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FOREWORD

The American Institute of Steel Construction, founded 1921, is the non-profit technical specifying and trade organization for the fabricated structural steel industry in the United States. Executive and engineering headquarters of AISC are maintained in Chicago.

The Institute is supported by four classes of membership: Active Members engaged in the fabrication, production, and sale of structural steel; Associate Members, who include Erectors, Detailers, Industry-Related Consultants, Software Developers, and Steel Product Manufacturers; Professional Members, who are individuals or firms engaged in the practice of architecture or engineering, including architectural and engineering educators; and Affiliate Members, who include General Contractors, Building Inspectors, and Code Officials. The continuing financial support and active participation of Members in the engineering, research and development activities of the Institute make possible the publishing of this *Steel Construction Manual.*

The Institute's objective is to make structural steel the material of choice, by being the leader in structural-steel-related technical and market-building activities, including: specification and code development, research, education, technical assistance, quality certification, standardization, and market development. AISC has a long tradition of service to the steel construction industry providing timely and reliable information.

To accomplish these objectives, the Institute publishes manuals, design guides, and specifications. Best known and most widely used is the *Steel Construction Manual*, which holds a highly respected position in engineering literature. Outstanding among AISC standards are the *Specification for Structural Steel Buildings* and the *Code of Standard Practice for Steel Buildings and Bridges.*

The Institute also publishes technical information and timely articles in its *Engineering Journal*, Design Guide series, *Modern Steel Construction* magazine, and other design aids, research reports, and journal articles. Almost all of the information AISC publishes is available for download from the AISC web site at www.aisc.org.

PREFACE

This Manual is the thirteenth major update of the AISC *Steel Construction Manual*, which was first published in 1927. With this revision, the previously separate Allowable Stress Design and Load and Resistance Factor Design methods have been combined. Thus, this Manual replaces both the 9th Edition ASD Manual and the 3rd Edition LRFD Manual. Much of the HSS Connections Manual has also been incorporated and updated in this Manual.

The following specifications, codes and standards are printed in Part 16 of this Manual:

- 2005 AISC *Specification for Structural Steel Buildings*
- 2004 RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*
- 2005 AISC *Code of Standard Practice for Steel Buildings and Bridges*

The following resources are also included on the CD included with this Manual:

- *AISC Design Examples*, which illustrates the application of tables and specification provisions that are included in this Manual.
- *AISC Shapes Database V13.0 and V13.0H*
- Background and supporting literature for the *AISC Steel Construction Manual*

The following major improvements have been made in this revision:

- The number of design examples has been expanded and included in a companion CD.
- All tabular information has been updated to comply with the *2005 Specification for Structural Buildings* and the standards and other documents referenced therein.
- Shape information has been updated to ASTM A6-05, including the new W36 shape series.
- Design methods have been delineated by making use of a dual-color format, with numbers indicated in blue type representing LRFD design values, and numbers indicated in green shading representing ASD design values. Tabulated values presented in black type are independent of design method.
- Information on HSS connections has been integrated throughout this Manual.
- W8 members have been reintegrated into design tables with cautionary statements regarding accessibility and dimensional constraints for connections made to them.
- Shapes with special design considerations, such as slenderness in compression or noncompactness in flexure, have been indicated throughout the member selection tables with footnotes.
- Workable flat dimensions of HSS members have been tabulated.
- Design properties for Pipe are now tabulated using the same wall thickness reduction factor used for HSS.
- An overview of provisions and a simplified method have been included for second-order analysis and stability requirements.
- New information has been added on corrosion protection and compatibility of dissimilar metals.
- Charts have been added for shear strength of plate girders.
- Lower-bound strengths for eccentrically loaded single angles have been tabulated.
- Tables have been added for the critical buckling stress of compression members.

- Tables for members subjected to combined axial load and bending have been expanded and improved.
- A table has been provided for calculating the strength of concentrically loaded weld groups.
- A direct calculation method has been added for calculating the buckling strength of double-coped members.
- Prying action provisions have been modified so that the tensile strength is used in the calculation rather than the yield strength.
- Beam bearing constants have been expanded to include all crippling and yielding cases.
- Revised design procedures for single-plate shear connections have been adopted, including a new design procedure for extended single-plate shear connections.
- An updated design procedure for moment end-plate connections has been adopted based upon yield-line analysis.
- The uniform force method weld ductility factor has changed from 1.4 to 1.25.
- Guidance on washer selection for anchor rods has been expanded.
- The design of bracket plates has been modified so that the plastic section modulus is used rather than the elastic section modulus.
- The AISC Design Guide Series and other supporting references have been further integrated through indexing and references to this material, where appropriate.

In addition, many other improvements have been made throughout this Manual.

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SCOPE

The dimensions and properties for structural products commonly used in steel building design and construction are given in this Part. For availability and proper material specifications for these products, as well as general specification requirements and other design considerations, see Part 2. For the design of members, see Parts 3 through 6. For the design of connections, see Parts 7 through 15. For AISC Specifications and Codes, see Part 16. For other miscellaneous information, see Part 17. For torsional and flexural-torsional properties of rolled shapes see AISC Design Guide 9, *Torsional Analysis of Structural Steel Members*. For surface areas, box perimeters and areas, *W*/*D* ratios and *A*/*D* ratios, see AISC Design Guide 19, *Fire Resistance of Structural Steel Framing*.

STRUCTURAL PRODUCTS

W-, M-, S-, and HP-Shapes

Four types of H-shaped (or I-shaped) members are covered in this Manual:

- W-shapes, which have essentially parallel inner and outer flange surfaces.
- M-shapes, which are H-shaped members that are not classified in ASTM A6 as W-, S-, or HP-shapes. M-shapes may have a sloped inside flange face or other cross-section features that do not meet the criteria for W-, S-, or HP-shapes.
- S-shapes (also known as American standard beams), which have a slope of approximately 16²/₃ percent (2 on 12) on the inner flange surfaces.
- HP-shapes (also known as bearing piles), which are similar to W-shapes, except their webs and flanges are of equal thickness and the depth and flange width are nominally equal for a given designation.

These shapes are designated by the mark W, M, S or HP, nominal depth (in.) and nominal weight (lb/ft). For example, a W24×55 is a W-shape that is nominally 24 in. deep and weighs 55 lb/ft.

The following dimensional and property information is given in this Manual for the W-, M-, S-, and HP-shapes covered in ASTM A6:

- Design dimensions, detailing dimensions, axial properties, and flexural properties are given in Tables 1–1, 1–2, 1–3, and 1–4 for W-, M-, S-, and HP-shapes, respectively.
- SI-equivalent designations are given in Table 17–1 for W-shapes and in Table 17–2 for M-, S-, and HP-shapes.

Tabulated decimal values are appropriate for use in design calculations, whereas fractional values are appropriate for use in detailing. All decimal and fractional values are similar with one exception: Because of the variation in fillet sizes used in shape production, the decimal value, k_{des}, is conservatively presented based on the smallest fillet used in production, and the fractional value, k_{det}, is conservatively presented based on the largest fillet used in production. For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

When appropriate, this Manual presents tabulated values for the Workable Gage of a section. The term Workable Gage refers to the gage for fasteners in the flange that provides for entering and tightening clearances and edge distance and spacing requirements. When the listed value is footnoted, the actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility. Other gages that provide for entering and tightening clearances and edge distance and spacing requirements can also be used.

Channels

Two types of channels are covered in this Manual:

- C-shapes (also known as American standard channels), which have a slope of approximately $16^2/_3$ percent (2 on 12) on the inner flange surfaces.
- MC-shapes (also known as miscellaneous channels), which have a slope other than $16^2/_3$ percent (2 on 12) on the inner flange surfaces.

These shapes are designated by the mark C or MC, nominal depth (in.) and nominal weight (lb/ft). For example, a C12×25 is a C-shape that is nominally 12 in. deep and weighs 25 lb/ft.

The following dimensional and property information is given in this Manual for the channels covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, flexural, and torsional properties are given in Tables 1–5 and 1–6 for C- and MC-shapes, respectively.
- SI-equivalent designations are given in Table 17–3.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Angles

Angles (also known as L-shapes) have legs of equal thickness and either equal or unequal leg sizes. Angles are designated by the mark L, leg sizes (in.) and thickness (in.). For example, an L4×3×$^1/_2$ is an angle with one 4-in. leg, one 3-in. leg, and $^1/_2$-in. thickness.

The following dimensional and property information is given in this Manual for the angles covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, flexural, and flexural-torsional properties are given in Table 1–7. The effects of leg-to-leg and toe fillet radii have been considered in the determination of these section properties. Workable gages on angle legs are tabulated at the end of Table 1–7.
- SI-equivalent designations are given in Table 17–4.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Structural Tees (WT-, MT-, and ST-Shapes)

Three types of structural tees are covered in this Manual:

- WT-shapes, which are made from W-shapes.
- MT-shapes, which are made from M-shapes.
- ST-shapes, which are made from S-shapes.

These shapes are designated by the mark WT, MT, or ST, nominal depth (in.) and nominal weight (lb/ft). WT-, MT-, and ST-shapes are split (sheared or thermal-cut) from W-, M-, and S-shapes, respectively, and have half the nominal depth and weight of that shape. For example, a WT12×27.5 is a structural tee split from a W-shape (W24×55), is nominally 12 in. deep and weighs 27.5 lb/ft. Although off-center splitting or splitting on two lines can be obtained by special order, the resulting nonstandard shape is not covered in this Manual.

The following dimensional and property information is given in this Manual for the structural tees cut from the W-, M-, and S-shapes covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, flexural, and torsional properties are given in Tables 1–8, 1–9, and 1–10 for WT-, MT-, and ST-shapes, respectively.
- SI-equivalent designations are given in Table 17–5 for WT-shapes and in Table 17–6 for MT- and ST-shapes.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Hollow Structural Sections (HSS)

Three types of HSS are covered in this Manual:

- Rectangular HSS, which have an essentially rectangular cross-section, except for rounded corners, and uniform wall thickness, except at the weld seam(s).
- Square HSS, which have an essentially square cross-section, except for rounded corners, and uniform wall thickness, except at the weld seam(s).
- Round HSS, which have an essentially round cross-section and uniform wall thickness, except at the weld seam(s).

In each case, ASTM A500 covers only electric-resistance-welded (ERW) HSS with a maximum periphery of 64 in. The coverage of HSS in this Manual is similarly limited.

Rectangular HSS are designated by the mark "HSS," overall outside dimensions (in.), and wall thickness (in.), with all dimensions expressed as fractional numbers. For example, an HSS10×10×1/2 is nominally 10 in. by 10 in. with a 1/2-in. wall thickness. Round HSS are designated by the term "HSS," nominal outside diameter (in.) and wall thickness (in.) with both dimensions expressed to three decimal places. For example, an HSS10.000×0.500 is nominally 10 in. in diameter with a 1/2-in. nominal wall thickness.

Per AISC Specification Section B3.12, the wall thickness used in design, t_{des}, is taken as 0.93 times the nominal wall thickness, t_{nom}. The rationale for this requirement is explained in the corresponding Commentary Section B3.12.

In calculating the tabulated b/t and h/t ratios, the outside corner radii are taken as $1.5t_{des}$ for rectangular and square HSS, per AISC Specification Section B4.2. In other tabulated design dimensions, the corner radii are taken as $2t_{des}$. In the tabulated workable flat dimensions of rectangular (and square) HSS, the outside corner radii are taken as $2.25t_{nom}$. The term Workable Flat refers to a reasonable flat width or depth of material for use in making connections to HSS. The workable flat dimension is provided as a reflection of current industry practice, although the tolerances of ASTM A500 allow a greater maximum corner radius of $3t_{des}$.

The following dimensional and property information is given in this Manual for the HSS covered in ASTM A500, A501, A618 or A847:

- Design dimensions, detailing dimensions, and axial, strong-axis flexural, weak-axis flexural, torsional and flexural-torsional properties are given in Tables 1–11 and 1–12 for rectangular and square HSS, respectively.
- Design dimensions, detailing dimensions, and axial, flexural, and torsional properties are given in Table 1–13 for round HSS.
- SI-equivalent designations are given in Tables 17–7, 17–8, and 17–9 for rectangular, square, and round HSS, respectively.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Pipe

Pipes have an essentially round cross-section and uniform thickness, except at the weld seam(s) for welded pipe.

Pipes up to and including NPS 12 are designated by the term "Pipe," nominal diameter (in.) and weight class (Std., x-strong, xx-strong). NPS stands for "nominal pipe size." For example, Pipe 5 Std. denotes a pipe with a 5-in. nominal diameter and a 0.258-in. wall thickness, which corresponds to the standard weight series. Pipes with wall thicknesses that do not correspond to the foregoing weight classes are designated by the term "Pipe," outside diameter (in.), and wall thickness (in.) with both expressed to three decimal places. For example, Pipe 14.000×0.375 and Pipe 5.563×0.500 are proper designations.

Per AISC Specification Section B3.12, the wall thickness used in design, t_{des}, is taken as 0.93 times the nominal wall thickness, t_{nom}. The rationale for this requirement is explained in the corresponding Commentary Section B3.12.

The following dimensional and property information is given in this Manual for the pipes covered in ASTM A53:

- Design dimensions, detailing dimensions, and axial, flexural, and torsional properties are given in Table 1–14.
- SI-equivalent designations are given in Table 17–10.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Double Angles

Double angles (also known as 2L-shapes) are made with two angles that are interconnected through their back-to-back legs along the length of the member, either in contact for the full length or separated by spacers at the points of interconnection.

These shapes are designated by the mark 2L, the sizes and thickness of their legs (in.), and their orientation when the angle legs are not of equal size (LLBB or SLBB).[1] For example, a 2L4×3×$^1/_2$ LLBB has two angles with one 4-in. leg and one 3-in. leg and the 4-in. legs are back-to-back; a 2L4×3×$^1/_2$ SLBB is similar, except the 3-in. legs are back-to-back. In both cases, the legs are $^1/_2$ in. thick.

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[1] LLBB stands for long legs back-to-back. SLBB stands for short legs back-to-back. Alternatively, the orientations LLV and SLV, which stand for long legs vertical and short legs vertical, respectively, can be used.

The following dimensional and property information is given in this Manual for the double angles built-up from the angles covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, strong-axis flexural, weak-axis flexural, torsional, and flexural-torsional properties are given in Table 1–15 for equal-leg, LLBB and SLBB angles. In each case, angle separations of zero in., 3/8 in., and 3/4 in. are covered. The effects of leg-to-leg and toe fillet radii have been considered in the determination of these section properties. For workable gages on legs of angles, see Table 1–7.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Double Channels

Double channels (also known as 2C- and 2MC-shapes) are made with two channels that are interconnected through their back-to-back webs along the length of the member, either in contact for the full length or separated by spacers at the points of interconnection.

These shapes are designated by the mark 2C or 2MC, nominal depth (in.), and nominal weight per channel (lb/ft). For example, a 2C12×25 is a double channel that consists of two channels that are each nominally 12 in. deep and each weigh 25 lb/ft.

The following dimensional and property information is given in this Manual for the double channels built-up from the channels covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, strong-axis flexural, and weak-axis flexural properties are given in Tables 1–16 and 1–17 for 2C- and 2MC-shapes, respectively. In each case, channel separations of zero, 3/8 in., and 3/4 in. are covered.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

W-Shapes and S-Shapes with Cap Channels

Common combined sections made with W- or S-shapes and channels (C- or MC-shapes) are tabulated in this Manual. In either case, the channel web is interconnected to the W-shape or S-shape top flange, respectively, with the flange toes down. The interconnection of the two elements must be designed for the horizontal shear, q, where

$$q = \frac{VQ}{I}$$

where

q = horizontal shear, kips/in.
V = vertical shear, kips.
Q = first moment of the channel area about the neutral axis of the combined cross section, in.^{3}
I = moment of inertia of the combined cross-section, in.^{4}

The effects of other forces, such as crane horizontal and lateral forces, may also require consideration, when applicable.

The following dimensional and property information is given in this Manual for combined sections, built-up from the W-shapes, S-shapes, and cap channels covered in ASTM A6:

- Design dimensions, detailing dimensions, and axial, strong-axis flexural and weak-axis flexural properties of W-shapes with cap channels are given in Table 1–19.
- Design dimensions, detailing dimensions, and axial, strong-axis flexural and weak-axis flexural properties of S-shapes with cap channels are given in Table 1–20.

For the definitions of the tabulated variables, refer to the Nomenclature section at the back of this Manual.

Plate Products

Plate products may be ordered as sheet, strip, or bar material. Sheet and strip are distinguished from structural bars and plates by their dimensional characteristics, as outlined in Table 2–2.

The historical classification system for structural bars and plates suggests that there is only a physical difference between them based upon size and production procedure. In raw form, flat stock has historically been classified as a bar if it is less than or equal to 8 in. wide and as a plate if it is greater than 8 in. wide. Bars are rolled between horizontal and vertical rolls and trimmed to length by shearing or thermal cutting on the ends only. Plates are generally produced using one of two methods:

1. Sheared plates are rolled between horizontal rolls and trimmed to width and length by shearing or thermal cutting on the edges and ends; or
2. Stripped plates are sheared or thermal cut from wider sheared plates.

There is very little, if any, structural difference between plates and bars. Consequently, the term “plate” is becoming a universally applied term today and a PL$^1/_2$×4$^1/_2$×1′-3″, for example, might be fabricated from plate or bar stock.

For structural plates, the preferred practice is to specify thickness in $^1/_{16}$-in. increments up to $^3/_8$-in. thickness, $^1/_8$-in. increments over $^3/_8$-in. to 1-in. thickness, and $^1/_4$-in. increments over 1-in. thickness. The current extreme widths for sheared plates is 200 in. Because mill practice regarding plate widths vary, individual mills should be consulted to determine preferences.

For bars, the preferred practice is to specify width in $^1/_4$-in. increments, and thickness and diameter in $^1/_8$-in. increments.

Raised-Pattern Floor Plates

Weights of raised-pattern floor plates are given in Table 1–18. Raised-pattern floor plates are commonly available in widths up to 120 in. For larger plate widths, see literature available from floor plate producers.

Crane Rails

Although crane rails are not listed as structural steel in Code of Standard Practice Section 2.1, this information is provided because some fabricators may choose to provide crane rails. Crane rails are designated by unit weight in lb/yard. Dimensions and properties for the crane rails shown are given in Table 1–21. Crane rails can be either heat treated or end hardened to reduce wear. For additional information or for profiles and properties of crane rails not listed, manufacturer’s catalogs should be consulted. For crane-rail connections, see Part 15.

Other Structural Products

The following other structural products are covered in this Manual as indicated:

- High-strength bolts, common bolts, washers, nuts, and direct-tension-indicator washers are covered in Part 7.
- Welding filler metals and fluxes are covered in Part 8.
- Forged steel structural hardware items, such as clevises, turnbuckles, sleeve nuts, recessed-pin nuts, and cotter pins are covered in Part 15.
- Anchor rods and threaded rods are covered in Part 14.

STANDARD MILL PRACTICES

The production of structural products is subject to unavoidable variations relative to the theoretical dimensions and profiles, due to many factors, including roll wear, roll dressing practices, and temperature effects. Such variations are limited by the dimensional and profile tolerances as summarized below.

Hot-Rolled Structural Shapes

Acceptable dimensional tolerances for hot-rolled structural shapes (W-, M-, S-, and HP-shapes), channels (C- and MC-shapes), and angles are given in ASTM A6 Section 13 and summarized in Tables 1–22 through 1–26. Supplementary information, including permissible variations for sheet and strip and for other grades of steel, can also be found in literature from steel plate producers and the Association of Iron and Steel Technology.

Hollow Structural Sections

Acceptable dimensional tolerances for HSS are given in ASTM A500 Section 10, A501 Section 11, A618 Section 8, or A847 Section 10, as applicable, and summarized in Tables 1–27 and 1–28, for rectangular and round HSS, respectively. Supplementary information can also be found in literature from HSS producers and the Steel Tube Institute, such as *Recommended Methods to Check Dimensional Tolerances on Hollow Structural Sections (HSS) Made to ASTM A500.*

Pipe

Acceptable dimensional tolerances for pipes are given in ASTM A53 Section 12 and summarized in Table 1–28. Supplementary information can also be found in literature from pipe producers.

Plate Products

Acceptable dimensional tolerances for plate products are given in ASTM A6 Section 13 and summarized in Table 1–29. Note that plate thickness can be specified in inches or by weight per square foot, and separate tolerances apply to each method. No decimal edge thickness can be assured for plate specified by the latter method. Supplementary information, including permissible variations for sheet and strip and for other grades of steel, can also be found in literature from steel plate producers and the Association of Iron and Steel Technology.

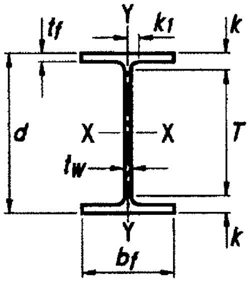


Table 1-1
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W44×335[c]	98.5	44.0	44	1.03	1	1/2	15.9	16	1.77	1 3/4	2.56	2 5/8	1 5/16	38 3/4	5 1/2
×290[c]	85.4	43.6	43 5/8	0.865	7/8	7/16	15.8	15 7/8	1.58	1 9/16	2.36	2 7/16	1 1/4	↓	↓
×262[c]	76.9	43.3	43 1/4	0.785	13/16	7/16	15.8	15 3/4	1.42	1 7/16	2.20	2 1/4	1 3/16	↓	↓
×230[c,v]	67.7	42.9	42 7/8	0.710	11/16	3/8	15.8	15 3/4	1.22	1 1/4	2.01	2 1/16	1 3/16	↓	↓
W40×593[h]	174	43.0	43	1.79	1 13/16	15/16	16.7	16 3/4	3.23	3 1/4	4.41	4 1/2	2 1/8	34	7 1/2
×503[h]	148	42.1	42	1.54	1 9/16	13/16	16.4	16 3/8	2.76	2 3/4	3.94	4	2	↓	↓
×431[h]	127	41.3	41 1/4	1.34	1 5/16	11/16	16.2	16 1/4	2.36	2 3/8	3.54	3 5/8	1 7/8	↓	↓
×397[h]	117	41.0	41	1.22	1 1/4	5/8	16.1	16 1/8	2.20	2 3/16	3.38	3 1/2	1 13/16	↓	↓
×372[h]	109	40.6	40 5/8	1.16	1 3/16	5/8	16.1	16 1/8	2.05	2 1/16	3.23	3 5/16	1 13/16	↓	↓
×362[h]	107	40.6	40 1/2	1.12	1 1/8	9/16	16.0	16	2.01	2	3.19	3 1/4	1 3/4	↓	↓
×324	95.3	40.2	40 1/8	1.00	1	1/2	15.9	15 7/8	1.81	1 13/16	2.99	3 1/16	1 11/16	↓	↓
×297[c]	87.4	39.8	39 7/8	0.930	15/16	1/2	15.8	15 7/8	1.65	1 5/8	2.83	2 15/16	1 11/16	↓	↓
×277[c]	81.4	39.7	39 3/4	0.830	13/16	7/16	15.8	15 7/8	1.58	1 9/16	2.76	2 7/8	1 5/8	↓	↓
×249[c]	73.3	39.4	39 3/8	0.750	3/4	3/8	15.8	15 3/4	1.42	1 7/16	2.60	2 11/16	1 9/16	↓	↓
×215[c]	63.4	39.0	39	0.650	5/8	5/16	15.8	15 3/4	1.22	1 1/4	2.40	2 1/2	1 9/16	↓	↓
×199[c]	58.5	38.7	38 5/8	0.650	5/8	5/16	15.8	15 3/4	1.07	1 1/16	2.25	2 5/16	1 9/16	↓	↓
W40×392[h]	115	41.6	41 5/8	1.42	1 7/16	3/4	12.4	12 3/8	2.52	2 1/2	3.70	3 13/16	1 15/16	34	7 1/2
×331[h]	97.5	40.8	40 3/4	1.22	1 1/4	5/8	12.2	12 1/8	2.13	2 1/8	3.31	3 3/8	1 13/16	↓	↓
×327[h]	96.0	40.8	40 3/4	1.18	1 3/16	5/8	12.1	12 1/8	2.13	2 1/8	3.31	3 3/8	1 13/16	↓	↓
×294	86.3	40.4	40 3/8	1.06	1 1/16	9/16	12.0	12	1.93	1 15/16	3.11	3 3/16	1 3/4	↓	↓
×278	82.0	40.2	40 1/8	1.03	1	1/2	12.0	12	1.81	1 13/16	2.99	3 1/16	1 3/4	↓	↓
×264	77.6	40.0	40	0.960	15/16	1/2	11.9	11 7/8	1.73	1 3/4	2.91	3	1 11/16	↓	↓
×235[c]	69.0	39.7	39 3/4	0.830	13/16	7/16	11.9	11 7/8	1.58	1 9/16	2.76	2 7/8	1 5/8	↓	↓
×211[c]	62.0	39.4	39 3/8	0.750	3/4	3/8	11.8	11 3/4	1.42	1 7/16	2.60	2 11/16	1 9/16	↓	↓
×183[c]	53.3	39.0	39	0.650	5/8	5/16	11.8	11 3/4	1.20	1 3/16	2.38	2 1/2	1 9/16	↓	↓
×167[c]	49.2	38.6	38 5/8	0.650	5/8	5/16	11.8	11 3/4	1.03	1	2.21	2 5/16	1 9/16	↓	↓
×149[c,v]	43.8	38.2	38 1/4	0.630	5/8	5/16	11.8	11 3/4	0.830	13/16	2.01	2 1/8	1 1/2	↓	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

Table 1-1 (continued)
W Shapes
Properties

W44 – W40

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
335	4.50	38.0	31100	1410	17.8	1620	1200	150	3.49	236	4.24	42.3	74.7	535000
290	5.02	45.0	27000	1240	17.8	1410	1040	132	3.49	205	4.21	42.0	50.9	461000
262	5.57	49.6	24100	1110	17.7	1270	923	117	3.47	182	4.17	41.9	37.3	405000
230	6.45	54.8	20800	971	17.5	1100	796	101	3.43	157	4.13	41.7	24.9	346000
593	2.58	19.1	50400	2340	17.0	2760	2520	302	3.80	481	4.63	39.8	445	997000
503	2.98	22.3	41600	1980	16.8	2310	2040	249	3.72	394	4.50	39.3	277	789000
431	3.44	25.5	34800	1690	16.6	1960	1690	208	3.65	328	4.41	38.9	177	638000
397	3.66	28.0	32000	1560	16.6	1800	1540	191	3.64	300	4.37	38.8	142	579000
372	3.93	29.5	29600	1460	16.5	1680	1420	177	3.60	277	4.34	38.6	116	528000
362	3.99	30.5	28900	1420	16.5	1640	1380	173	3.60	270	4.33	38.5	109	513000
324	4.40	34.2	25600	1280	16.4	1460	1220	153	3.58	239	4.28	38.4	79.4	448000
297	4.80	36.8	23200	1170	16.3	1330	1090	138	3.54	215	4.23	38.2	61.2	399000
277	5.03	41.2	21900	1100	16.4	1250	1040	132	3.58	204	4.25	38.1	51.5	379000
249	5.55	45.6	19600	993	16.3	1120	926	118	3.55	182	4.21	38.0	38.1	334000
215	6.45	52.6	16700	859	16.2	964	796	101	3.54	156	4.18	37.8	24.8	284000
199	7.39	52.6	14900	770	16.0	869	695	88.2	3.45	137	4.12	37.6	18.3	246000
392	2.45	24.1	29900	1440	16.1	1710	803	130	2.64	212	3.30	39.1	172	306000
331	2.86	28.0	24700	1210	15.9	1430	644	106	2.57	172	3.21	38.7	105	241000
327	2.85	29.0	24500	1200	16.0	1410	640	105	2.58	170	3.21	38.7	103	239000
294	3.11	32.2	21900	1080	15.9	1270	562	93.5	2.55	150	3.16	38.5	76.6	208000
278	3.31	33.3	20500	1020	15.8	1190	521	87.1	2.52	140	3.13	38.4	65.0	192000
264	3.45	35.6	19400	971	15.8	1130	493	82.6	2.52	132	3.12	38.3	56.1	181000
235	3.77	41.2	17400	875	15.9	1010	444	74.6	2.54	118	3.11	38.1	41.3	161000
211	4.17	45.6	15500	786	15.8	906	390	66.1	2.51	105	3.07	38.0	30.4	141000
183	4.92	52.6	13200	675	15.7	774	331	56.0	2.49	88.3	3.04	37.8	19.3	118000
167	5.76	52.6	11600	600	15.3	693	283	47.9	2.40	76.0	2.98	37.6	14.0	99700
149	7.11	54.3	9800	513	15.0	598	229	38.8	2.29	62.2	2.89	37.4	9.36	80000

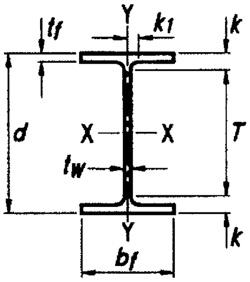


Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, *A*	Depth, *d*		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W36×800^{h}	236	42.6	42 1/2	2.38	2 3/8	1 3/16	18.0	18	4.29	4 5/16	5.24	5 9/16	2 3/8	31 3/8	7 1/2
×652^{h}	192	41.1	41	1.97	2	1	17.6	17 5/8	3.54	3 9/16	4.49	4 13/16	2 3/16		
×529^{h}	156	39.8	39 3/4	1.61	1 5/8	13/16	17.2	17 1/4	2.91	2 15/16	3.86	4 3/16	2		
×487^{h}	143	39.3	39 3/8	1.50	1 1/2	3/4	17.1	17 1/8	2.68	2 11/16	3.63	4	1 15/16		
×441^{h}	130	38.9	38 7/8	1.36	1 3/8	11/16	17.0	17	2.44	2 7/16	3.39	3 3/4	1 7/8		
×395^{h}	116	38.4	38 3/8	1.22	1 1/4	5/8	16.8	16 7/8	2.20	2 3/16	3.15	3 7/16	1 13/16		
×361^{h}	106	38.0	38	1.12	1 1/8	9/16	16.7	16 3/4	2.01	2	2.96	3 5/16	1 3/4		
×330	97.0	37.7	37 5/8	1.02	1	1/2	16.6	16 5/8	1.85	1 7/8	2.80	3 1/8	1 3/4		
×302	88.8	37.3	37 3/8	0.945	15/16	1/2	16.7	16 5/8	1.68	1 11/16	2.63	3	1 11/16		
×282^{c}	82.9	37.1	37 1/8	0.885	7/8	7/16	16.6	16 5/8	1.57	1 9/16	2.52	2 7/8	1 5/8		
×262^{c}	77.0	36.9	36 7/8	0.840	13/16	7/16	16.6	16 1/2	1.44	1 7/16	2.39	2 3/4	1 5/8		
×247^{c}	72.5	36.7	36 5/8	0.800	13/16	7/16	16.5	16 1/2	1.35	1 3/8	2.30	2 5/8	1 5/8		
×231^{c}	68.1	36.5	36 1/2	0.760	3/4	3/8	16.5	16 1/2	1.26	1 1/4	2.21	2 9/16	1 9/16		
W36×256	75.4	37.4	37 3/8	0.960	15/16	1/2	12.2	12 1/4	1.73	1 3/4	2.48	2 5/8	1 5/16	32 1/8	5 1/2
×232^{c}	68.1	37.1	37 1/8	0.870	7/8	7/16	12.1	12 1/8	1.57	1 9/16	2.32	2 7/16	1 1/4		
×210^{c}	61.8	36.7	36 3/4	0.830	13/16	7/16	12.2	12 1/8	1.36	1 3/8	2.11	2 5/16	1 1/4		
×194^{c}	57.0	36.5	36 1/2	0.765	3/4	3/8	12.1	12 1/8	1.26	1 1/4	2.01	2 3/16	1 3/16		
×182^{c}	53.6	36.3	36 3/8	0.725	3/4	3/8	12.1	12 1/8	1.18	1 3/16	1.93	2 1/8	1 3/16		
×170^{c}	50.1	36.2	36 1/8	0.680	11/16	3/8	12.0	12	1.10	1 1/8	1.85	2	1 3/16		
×160^{c}	47.0	36.0	36	0.650	5/8	5/16	12.0	12	1.02	1	1.77	1 15/16	1 1/8		
×150^{c}	44.2	35.9	35 7/8	0.625	5/8	5/16	12.0	12	0.940	15/16	1.69	1 7/8	1 1/8		
×135^{c,v}	39.7	35.6	35 1/2	0.600	5/8	5/16	12.0	12	0.790	13/16	1.54	1 11/16	1 1/8		
W33×387^{h}	114	36.0	36	1.26	1 1/4	5/8	16.2	16 1/4	2.28	2 1/4	3.07	3 3/16	1 7/16	29 5/8	5 1/2
×354^{h}	104	35.6	35 1/2	1.16	1 3/16	5/8	16.1	16 1/8	2.09	2 1/16	2.88	2 15/16	1 3/8		
×318	93.6	35.2	35 1/8	1.04	1 1/16	9/16	16.0	16	1.89	1 7/8	2.68	2 3/4	1 5/16		
×291	85.7	34.8	34 7/8	0.960	15/16	1/2	15.9	15 7/8	1.73	1 3/4	2.52	2 5/8	1 5/16		
×263	77.5	34.5	34 1/2	0.870	7/8	7/16	15.8	15 3/4	1.57	1 9/16	2.36	2 7/16	1 1/4		
×241^{c}	71.0	34.2	34 1/8	0.830	13/16	7/16	15.9	15 7/8	1.40	1 3/8	2.19	2 1/4	1 1/4		
×221^{c}	65.2	33.9	33 7/8	0.775	3/4	3/8	15.8	15 3/4	1.28	1 1/4	2.06	2 1/8	1 3/16		
×201^{c}	59.2	33.7	33 5/8	0.715	11/16	3/8	15.7	15 3/4	1.15	1 1/8	1.94	2	1 3/16		
W33×169^{c}	49.5	33.8	33 7/8	0.670	11/16	3/8	11.5	11 1/2	1.22	1 1/4	1.92	2 1/8	1 3/16	29 5/8	5 1/2
×152^{c}	44.8	33.5	33 1/2	0.635	5/8	5/16	11.6	11 5/8	1.06	1 1/16	1.76	1 15/16	1 1/8		
×141^{c}	41.6	33.3	33 1/4	0.605	5/8	5/16	11.5	11 1/2	0.960	15/16	1.66	1 13/16	1 1/8		
×130^{c}	38.3	33.1	33 1/8	0.580	9/16	5/16	11.5	11 1/2	0.855	7/8	1.56	1 3/4	1 1/8		
×118^{c,v}	34.7	32.9	32 7/8	0.550	9/16	5/16	11.5	11 1/2	0.740	3/4	1.44	1 5/8	1 1/8		

[c] Shape is slender for compression with $F_y = 50$ ksi.
[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.
[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

Table 1–1 (continued)
W Shapes
Properties

W36 - W33

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
800	2.10	13.5	64700	3040	16.6	3650	4200	467	4.22	743	5.14	38.3	1060	1540000
652	2.48	16.3	50600	2460	16.2	2910	3230	367	4.10	581	4.96	37.5	593	1130000
529	2.96	19.9	39600	1990	16.0	2330	2490	289	4.00	454	4.80	36.9	327	846000
487	3.19	21.4	36000	1830	15.8	2130	2250	263	3.96	412	4.74	36.7	258	754000
441	3.48	23.6	32100	1650	15.7	1910	1990	235	3.92	368	4.69	36.4	194	661000
395	3.83	26.3	28500	1490	15.7	1710	1750	208	3.88	325	4.61	36.2	142	575000
361	4.16	28.6	25700	1350	15.6	1550	1570	188	3.85	293	4.58	36.0	109	509000
330	4.49	31.4	23300	1240	15.5	1410	1420	171	3.83	265	4.53	35.8	84.3	456000
302	4.96	33.9	21100	1130	15.4	1280	1300	156	3.82	241	4.53	35.7	64.3	412000
282	5.29	36.2	19600	1050	15.4	1190	1200	144	3.80	223	4.50	35.5	52.7	378000
262	5.75	38.2	17900	972	15.3	1100	1090	132	3.76	204	4.46	35.4	41.6	342000
247	6.11	40.1	16700	913	15.2	1030	1010	123	3.74	190	4.42	35.3	34.7	316000
231	6.54	42.2	15600	854	15.1	963	940	114	3.71	176	4.40	35.2	28.7	292000
256	3.53	33.8	16800	895	14.9	1040	528	86.5	2.65	137	3.25	35.7	52.9	168000
232	3.86	37.3	15000	809	14.8	936	468	77.2	2.62	122	3.21	35.6	39.6	148000
210	4.48	39.1	13200	719	14.6	833	411	67.5	2.58	107	3.18	35.3	28.0	128000
194	4.81	42.4	12100	664	14.6	767	375	61.9	2.56	97.7	3.15	35.2	22.2	116000
182	5.12	44.8	11300	623	14.5	718	347	57.6	2.55	90.7	3.13	35.2	18.5	107000
170	5.47	47.7	10500	581	14.5	668	320	53.2	2.53	83.8	3.11	35.1	15.1	98500
160	5.88	49.9	9760	542	14.4	624	295	49.1	2.50	77.3	3.08	35.0	12.4	90200
150	6.37	51.9	9040	504	14.3	581	270	45.1	2.47	70.9	3.06	34.9	10.1	82200
135	7.56	54.1	7800	439	14.0	509	225	37.7	2.38	59.7	2.99	34.8	7.00	68100
387	3.55	23.7	24300	1350	14.6	1560	1620	200	3.77	312	4.49	33.7	148	459000
354	3.85	25.7	22000	1240	14.5	1420	1460	181	3.74	282	4.44	33.5	115	408000
318	4.23	28.7	19500	1110	14.5	1270	1290	161	3.71	250	4.39	33.3	84.4	357000
291	4.60	31.0	17700	1020	14.4	1160	1160	146	3.68	226	4.35	33.1	65.1	319000
263	5.03	34.3	15900	919	14.3	1040	1040	131	3.66	202	4.31	33.0	48.7	281000
241	5.66	35.9	14200	831	14.1	940	933	118	3.62	182	4.29	32.8	36.2	251000
221	6.20	38.5	12900	759	14.1	857	840	106	3.59	164	4.25	32.7	27.8	224000
201	6.85	41.7	11600	686	14.0	773	749	95.2	3.56	147	4.21	32.5	20.8	198000
169	4.71	44.7	9290	549	13.7	629	310	53.9	2.50	84.4	3.03	32.6	17.7	82400
152	5.48	47.2	8160	487	13.5	559	273	47.2	2.47	73.9	3.01	32.4	12.4	71700
141	6.01	49.6	7450	448	13.4	514	246	42.7	2.43	66.9	2.98	32.3	9.70	64400
130	6.73	51.7	6710	406	13.2	467	218	37.9	2.39	59.5	2.94	32.2	7.37	56600
118	7.76	54.5	5900	359	13.0	415	187	32.6	2.32	51.3	2.89	32.1	5.30	48300

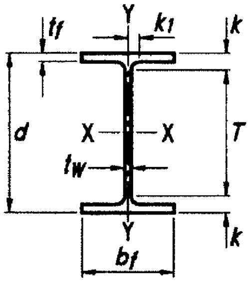


Table 1-1 (continued)
W Shapes
Dimensions

				Web			Flange				Distance				
Shape	Area, A	Depth, d		Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W30×391^h	115	33.2	$33\frac{1}{4}$	1.36	$1\frac{3}{8}$	$\frac{11}{16}$	15.6	$15\frac{5}{8}$	2.44	$2\frac{7}{16}$	3.23	$3\frac{3}{8}$	$1\frac{1}{2}$	$26\frac{1}{2}$	$5\frac{1}{2}$
×357^h	105	32.8	$32\frac{3}{4}$	1.24	$1\frac{1}{4}$	$\frac{5}{8}$	15.5	$15\frac{1}{2}$	2.24	$2\frac{1}{4}$	3.03	$3\frac{1}{8}$	$1\frac{7}{16}$	↓	↓
×326^h	95.8	32.4	$32\frac{3}{8}$	1.14	$1\frac{1}{8}$	$\frac{9}{16}$	15.4	$15\frac{3}{8}$	2.05	$2\frac{1}{16}$	2.84	$2\frac{15}{16}$	$1\frac{3}{8}$	↓	↓
×292	85.9	32.0	32	1.02	1	$\frac{1}{2}$	15.3	$15\frac{1}{4}$	1.85	$1\frac{7}{8}$	2.64	$2\frac{3}{4}$	$1\frac{5}{16}$	↓	↓
×261	76.9	31.6	$31\frac{5}{8}$	0.930	$\frac{15}{16}$	$\frac{1}{2}$	15.2	$15\frac{1}{8}$	1.65	$1\frac{5}{8}$	2.44	$2\frac{9}{16}$	$1\frac{5}{16}$	↓	↓
×235	69.2	31.3	$31\frac{1}{4}$	0.830	$\frac{13}{16}$	$\frac{7}{16}$	15.1	15	1.50	$1\frac{1}{2}$	2.29	$2\frac{3}{8}$	$1\frac{1}{4}$	↓	↓
×211	62.2	30.9	31	0.775	$\frac{3}{4}$	$\frac{3}{8}$	15.1	$15\frac{1}{8}$	1.32	$1\frac{5}{16}$	2.10	$2\frac{1}{4}$	$1\frac{3}{16}$	↓	↓
×191^c	56.3	30.7	$30\frac{5}{8}$	0.710	$\frac{11}{16}$	$\frac{3}{8}$	15.0	15	1.19	$1\frac{3}{16}$	1.97	$2\frac{1}{16}$	$1\frac{3}{16}$	↓	↓
×173^c	51.0	30.4	$30\frac{1}{2}$	0.655	$\frac{5}{8}$	$\frac{5}{16}$	15.0	15	1.07	$1\frac{1}{16}$	1.85	2	$1\frac{1}{8}$	↓	↓
W30×148^c	43.5	30.7	$30\frac{5}{8}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	10.5	$10\frac{1}{2}$	1.18	$1\frac{3}{16}$	1.83	$2\frac{1}{16}$	$1\frac{1}{8}$	$26\frac{1}{2}$	$5\frac{1}{2}$
×132^c	38.9	30.3	$30\frac{1}{4}$	0.615	$\frac{5}{8}$	$\frac{5}{16}$	10.5	$10\frac{1}{2}$	1.00	1	1.65	$1\frac{7}{8}$	$1\frac{1}{8}$	↓	↓
×124^c	36.5	30.2	$30\frac{1}{8}$	0.585	$\frac{9}{16}$	$\frac{5}{16}$	10.5	$10\frac{1}{2}$	0.930	$\frac{15}{16}$	1.58	$1\frac{13}{16}$	$1\frac{1}{8}$	↓	↓
×116^c	34.2	30.0	30	0.565	$\frac{9}{16}$	$\frac{5}{16}$	10.5	$10\frac{1}{2}$	0.850	$\frac{7}{8}$	1.50	$1\frac{3}{4}$	$1\frac{1}{8}$	↓	↓
×108^c	31.7	29.8	$29\frac{7}{8}$	0.545	$\frac{9}{16}$	$\frac{5}{16}$	10.5	$10\frac{1}{2}$	0.760	$\frac{3}{4}$	1.41	$1\frac{11}{16}$	$1\frac{1}{8}$	↓	↓
×99^c	29.1	29.7	$29\frac{5}{8}$	0.520	$\frac{1}{2}$	$\frac{1}{4}$	10.5	$10\frac{1}{2}$	0.670	$\frac{11}{16}$	1.32	$1\frac{9}{16}$	$1\frac{1}{16}$	↓	↓
×90^{c,v}	26.4	29.5	$29\frac{1}{2}$	0.470	$\frac{1}{2}$	$\frac{1}{4}$	10.4	$10\frac{3}{8}$	0.610	$\frac{5}{8}$	1.26	$1\frac{1}{2}$	$1\frac{1}{16}$	↓	↓
W27×539^h	159	32.5	$32\frac{1}{2}$	1.97	2	1	15.3	$15\frac{1}{4}$	3.54	$3\frac{9}{16}$	4.33	$4\frac{7}{16}$	$1\frac{13}{16}$	$23\frac{5}{8}$	$5\frac{1}{2}$^{g}
×368^h	108	30.4	$30\frac{3}{8}$	1.38	$1\frac{3}{8}$	$\frac{11}{16}$	14.7	$14\frac{5}{8}$	2.48	$2\frac{1}{2}$	3.27	$3\frac{3}{8}$	$1\frac{1}{2}$	↓	$5\frac{1}{2}$
×336^h	98.9	30.0	30	1.26	$1\frac{1}{4}$	$\frac{5}{8}$	14.6	$14\frac{1}{2}$	2.28	$2\frac{1}{4}$	3.07	$3\frac{3}{16}$	$1\frac{7}{16}$	↓	↓
×307^h	90.4	29.6	$29\frac{5}{8}$	1.16	$1\frac{3}{16}$	$\frac{5}{8}$	14.4	$14\frac{1}{2}$	2.09	$2\frac{1}{16}$	2.88	3	$1\frac{7}{16}$	↓	↓
×281	82.9	29.3	$29\frac{1}{4}$	1.06	$1\frac{1}{16}$	$\frac{9}{16}$	14.4	$14\frac{3}{8}$	1.93	$1\frac{15}{16}$	2.72	$2\frac{13}{16}$	$1\frac{3}{8}$	↓	↓
×258	76.0	29.0	29	0.980	1	$\frac{1}{2}$	14.3	$14\frac{1}{4}$	1.77	$1\frac{3}{4}$	2.56	$2\frac{11}{16}$	$1\frac{5}{16}$	↓	↓
×235	69.4	28.7	$28\frac{5}{8}$	0.910	$\frac{15}{16}$	$\frac{1}{2}$	14.2	$14\frac{1}{4}$	1.61	$1\frac{5}{8}$	2.40	$2\frac{1}{2}$	$1\frac{5}{16}$	↓	↓
×217	64.0	28.4	$28\frac{3}{8}$	0.830	$\frac{13}{16}$	$\frac{7}{16}$	14.1	$14\frac{1}{8}$	1.50	$1\frac{1}{2}$	2.29	$2\frac{3}{8}$	$1\frac{1}{4}$	↓	↓
×194	57.2	28.1	$28\frac{1}{8}$	0.750	$\frac{3}{4}$	$\frac{3}{8}$	14.0	14	1.34	$1\frac{5}{16}$	2.13	$2\frac{1}{4}$	$1\frac{3}{16}$	↓	↓
×178	52.5	27.8	$27\frac{3}{4}$	0.725	$\frac{3}{4}$	$\frac{3}{8}$	14.1	$14\frac{1}{8}$	1.19	$1\frac{3}{16}$	1.98	$2\frac{1}{16}$	$1\frac{3}{16}$	↓	↓
×161^c	47.6	27.6	$27\frac{5}{8}$	0.660	$\frac{11}{16}$	$\frac{3}{8}$	14.0	14	1.08	$1\frac{1}{16}$	1.87	2	$1\frac{3}{16}$	↓	↓
×146^c	43.1	27.4	$27\frac{3}{8}$	0.605	$\frac{5}{8}$	$\frac{5}{16}$	14.0	14	0.975	1	1.76	$1\frac{7}{8}$	$1\frac{1}{8}$	↓	↓
W27×129^c	37.8	27.6	$27\frac{5}{8}$	0.610	$\frac{5}{8}$	$\frac{5}{16}$	10.0	10	1.10	$1\frac{1}{8}$	1.70	2	$1\frac{1}{8}$	$23\frac{5}{8}$	$5\frac{1}{2}$
×114^c	33.5	27.3	$27\frac{1}{4}$	0.570	$\frac{9}{16}$	$\frac{5}{16}$	10.1	$10\frac{1}{8}$	0.930	$\frac{15}{16}$	1.53	$1\frac{13}{16}$	$1\frac{1}{8}$	↓	↓
×102^c	30.0	27.1	$27\frac{1}{8}$	0.515	$\frac{1}{2}$	$\frac{1}{4}$	10.0	10	0.830	$\frac{13}{16}$	1.43	$1\frac{3}{4}$	$1\frac{1}{16}$	↓	↓
×94^c	27.7	26.9	$26\frac{7}{8}$	0.490	$\frac{1}{2}$	$\frac{1}{4}$	10.0	10	0.745	$\frac{3}{4}$	1.34	$1\frac{5}{8}$	$1\frac{1}{16}$	↓	↓
×84^c	24.8	26.7	$26\frac{3}{4}$	0.460	$\frac{7}{16}$	$\frac{1}{4}$	10.0	10	0.640	$\frac{5}{8}$	1.24	$1\frac{9}{16}$	$1\frac{1}{16}$	↓	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

Table 1-1 (continued)
W Shapes
Properties

W30 - W27

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
391	3.19	19.7	20700	1250	13.4	1450	1550	198	3.67	310	4.37	30.8	173	366000
357	3.45	21.6	18700	1140	13.3	1320	1390	179	3.64	279	4.32	30.6	134	324000
326	3.75	23.4	16800	1040	13.2	1190	1240	162	3.60	252	4.27	30.4	103	287000
292	4.12	26.2	14900	930	13.2	1060	1100	144	3.58	223	4.22	30.2	75.2	250000
261	4.59	28.7	13100	829	13.1	943	959	127	3.53	196	4.16	30.0	54.1	215000
235	5.02	32.2	11700	748	13.0	847	855	114	3.51	175	4.13	29.8	40.3	190000
211	5.74	34.5	10300	665	12.9	751	757	100	3.49	155	4.10	29.6	28.4	166000
191	6.35	37.7	9200	600	12.8	675	673	89.5	3.46	138	4.07	29.5	21.0	146000
173	7.04	40.8	8230	541	12.7	607	598	79.8	3.42	123	4.03	29.4	15.6	129000
148	4.44	41.6	6680	436	12.4	500	227	43.3	2.28	68.0	2.77	29.5	14.5	49400
132	5.27	43.9	5770	380	12.2	437	196	37.2	2.25	58.4	2.75	29.3	9.72	42100
124	5.65	46.2	5360	355	12.1	408	181	34.4	2.23	54.0	2.73	29.2	7.99	38600
116	6.17	47.8	4930	329	12.0	378	164	31.3	2.19	49.2	2.70	29.2	6.43	34900
108	6.89	49.6	4470	299	11.9	346	146	27.9	2.15	43.9	2.66	29.1	4.99	30900
99	7.80	51.9	3990	269	11.7	312	128	24.5	2.10	38.6	2.62	29.0	3.77	26800
90	8.52	57.5	3610	245	11.7	283	115	22.1	2.09	34.7	2.60	28.9	2.84	24000
539	2.15	12.1	25600	1570	12.7	1890	2110	277	3.65	437	4.41	29.0	496	443000
368	2.96	17.3	16200	1060	12.2	1240	1310	179	3.48	279	4.14	27.9	170	255000
336	3.19	18.9	14600	972	12.1	1130	1180	162	3.45	252	4.09	27.7	131	226000
307	3.46	20.6	13100	887	12.0	1030	1050	146	3.41	227	4.04	27.5	101	199000
281	3.72	22.5	11900	814	12.0	936	953	133	3.39	206	4.00	27.4	79.5	178000
258	4.03	24.4	10800	745	11.9	852	859	120	3.36	187	3.96	27.2	61.6	159000
235	4.41	26.2	9700	677	11.8	772	769	108	3.33	168	3.92	27.1	47.0	141000
217	4.71	28.7	8910	627	11.8	711	704	100	3.32	154	3.89	26.9	37.6	128000
194	5.24	31.8	7860	559	11.7	631	619	88.1	3.29	136	3.85	26.8	27.1	111000
178	5.92	32.9	7020	505	11.6	570	555	78.8	3.25	122	3.83	26.6	20.1	98400
161	6.49	36.1	6310	458	11.5	515	497	70.9	3.23	109	3.79	26.5	15.1	87300
146	7.16	39.4	5660	414	11.5	464	443	63.5	3.20	97.7	3.76	26.4	11.3	77200
129	4.55	39.7	4760	345	11.2	395	184	36.8	2.21	57.6	2.66	26.5	11.1	32500
114	5.41	42.5	4080	299	11.0	343	159	31.5	2.18	49.3	2.64	26.4	7.33	27600
102	6.03	47.1	3620	267	11.0	305	139	27.8	2.15	43.4	2.62	26.3	5.28	24000
94	6.70	49.5	3270	243	10.9	278	124	24.8	2.12	38.8	2.59	26.2	4.03	21300
84	7.78	52.7	2850	213	10.7	244	106	21.2	2.07	33.2	2.54	26.1	2.81	17900

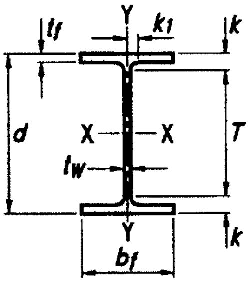


Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W24×370^{h}	109	28.0	28	1.52	$1\frac{1}{2}$	$\frac{3}{4}$	13.7	$13\frac{5}{8}$	2.72	$2\frac{3}{4}$	3.22	$3\frac{5}{8}$	$1\frac{9}{16}$	$20\frac{3}{4}$	$5\frac{1}{2}$
×335^{h}	98.4	27.5	$27\frac{1}{2}$	1.38	$1\frac{3}{8}$	$\frac{11}{16}$	13.5	$13\frac{1}{2}$	2.48	$2\frac{1}{2}$	2.98	$3\frac{3}{8}$	$1\frac{1}{2}$	↓	↓
×306^{h}	89.8	27.1	$27\frac{1}{8}$	1.26	$1\frac{1}{4}$	$\frac{5}{8}$	13.4	$13\frac{3}{8}$	2.28	$2\frac{1}{4}$	2.78	$3\frac{3}{16}$	$1\frac{7}{16}$	↓	↓
x279^{h}	82.0	26.7	$26\frac{3}{4}$	1.16	$1\frac{3}{16}$	$\frac{5}{8}$	13.3	$13\frac{1}{4}$	2.09	$2\frac{1}{16}$	2.59	3	$1\frac{7}{16}$	↓	↓
×250	73.5	26.3	$26\frac{3}{8}$	1.04	$1\frac{1}{16}$	$\frac{9}{16}$	13.2	$13\frac{1}{8}$	1.89	$1\frac{7}{8}$	2.39	$2\frac{13}{16}$	$1\frac{3}{8}$	↓	↓
×229	67.2	26.0	26	0.960	$\frac{15}{16}$	$\frac{1}{2}$	13.1	$13\frac{1}{8}$	1.73	$1\frac{3}{4}$	2.23	$2\frac{5}{8}$	$1\frac{5}{16}$	↓	↓
×207	60.7	25.7	$25\frac{3}{4}$	0.870	$\frac{7}{8}$	$\frac{7}{16}$	13.0	13	1.57	$1\frac{9}{16}$	2.07	$2\frac{1}{2}$	$1\frac{1}{4}$	↓	↓
×192	56.3	25.5	$25\frac{1}{2}$	0.810	$\frac{13}{16}$	$\frac{7}{16}$	13.0	13	1.46	$1\frac{7}{16}$	1.96	$2\frac{3}{8}$	$1\frac{1}{4}$	↓	↓
×176	51.7	25.2	$25\frac{1}{4}$	0.750	$\frac{3}{4}$	$\frac{3}{8}$	12.9	$12\frac{7}{8}$	1.34	$1\frac{5}{16}$	1.84	$2\frac{1}{4}$	$1\frac{3}{16}$	↓	↓
×162	47.7	25.0	25	0.705	$\frac{11}{16}$	$\frac{3}{8}$	13.0	13	1.22	$1\frac{1}{4}$	1.72	$2\frac{1}{8}$	$1\frac{3}{16}$	↓	↓
×146	43.0	24.7	$24\frac{3}{4}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.9	$12\frac{7}{8}$	1.09	$1\frac{1}{16}$	1.59	2	$1\frac{1}{8}$	↓	↓
×131	38.5	24.5	$24\frac{1}{2}$	0.605	$\frac{5}{8}$	$\frac{5}{16}$	12.9	$12\frac{7}{8}$	0.960	$\frac{15}{16}$	1.46	$1\frac{7}{8}$	$1\frac{1}{8}$	↓	↓
×117^{c}	34.4	24.3	$24\frac{1}{4}$	0.550	$\frac{9}{16}$	$\frac{5}{16}$	12.8	$12\frac{3}{4}$	0.850	$\frac{7}{8}$	1.35	$1\frac{3}{4}$	$1\frac{1}{8}$	↓	↓
×104^{c}	30.6	24.1	24	0.500	$\frac{1}{2}$	$\frac{1}{4}$	12.8	$12\frac{3}{4}$	0.750	$\frac{3}{4}$	1.25	$1\frac{5}{8}$	$1\frac{1}{16}$	↓	↓
W24×103^{c}	30.3	24.5	$24\frac{1}{2}$	0.550	$\frac{9}{16}$	$\frac{5}{16}$	9.00	9	0.980	1	1.48	$1\frac{7}{8}$	$1\frac{1}{8}$	$20\frac{3}{4}$	$5\frac{1}{2}$
×94^{c}	27.7	24.3	$24\frac{1}{4}$	0.515	$\frac{1}{2}$	$\frac{1}{4}$	9.07	$9\frac{1}{8}$	0.875	$\frac{7}{8}$	1.38	$1\frac{3}{4}$	$1\frac{1}{16}$	↓	↓
×84^{c}	24.7	24.1	$24\frac{1}{8}$	0.470	$\frac{1}{2}$	$\frac{1}{4}$	9.02	9	0.770	$\frac{3}{4}$	1.27	$1\frac{11}{16}$	$1\frac{1}{16}$	↓	↓
×76^{c}	22.4	23.9	$23\frac{7}{8}$	0.440	$\frac{7}{16}$	$\frac{1}{4}$	8.99	9	0.680	$\frac{11}{16}$	1.18	$1\frac{9}{16}$	$1\frac{1}{16}$	↓	↓
×68^{c}	20.1	23.7	$23\frac{3}{4}$	0.415	$\frac{7}{16}$	$\frac{1}{4}$	8.97	9	0.585	$\frac{9}{16}$	1.09	$1\frac{1}{2}$	$1\frac{1}{16}$	↓	↓
W24×62^{c}	18.2	23.7	$23\frac{3}{4}$	0.430	$\frac{7}{16}$	$\frac{1}{4}$	7.04	7	0.590	$\frac{9}{16}$	1.09	$1\frac{1}{2}$	$1\frac{1}{16}$	$20\frac{3}{4}$	$3\frac{1}{2}^{g}$
×55^{c,v}	16.2	23.6	$23\frac{5}{8}$	0.395	$\frac{3}{8}$	$\frac{3}{16}$	7.01	7	0.505	$\frac{1}{2}$	1.01	$1\frac{7}{16}$	1	$20\frac{3}{4}$	$3\frac{1}{2}^{g}$
W21×201	59.2	23.0	23	0.910	$\frac{15}{16}$	$\frac{1}{2}$	12.6	$12\frac{5}{8}$	1.63	$1\frac{5}{8}$	2.13	$2\frac{1}{2}$	$1\frac{5}{16}$	18	$5\frac{1}{2}$
×182	53.6	22.7	$22\frac{3}{4}$	0.830	$\frac{13}{16}$	$\frac{7}{16}$	12.5	$12\frac{1}{2}$	1.48	$1\frac{1}{2}$	1.98	$2\frac{3}{8}$	$1\frac{1}{4}$	↓	↓
×166	48.8	22.5	$22\frac{1}{2}$	0.750	$\frac{3}{4}$	$\frac{3}{8}$	12.4	$12\frac{3}{8}$	1.36	$1\frac{3}{8}$	1.86	$2\frac{1}{4}$	$1\frac{3}{16}$	↓	↓
×147	43.2	22.1	22	0.720	$\frac{3}{4}$	$\frac{3}{8}$	12.5	$12\frac{1}{2}$	1.15	$1\frac{1}{8}$	1.65	2	$1\frac{3}{16}$	↓	↓
×132	38.8	21.8	$21\frac{7}{8}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.4	$12\frac{1}{2}$	1.04	$1\frac{1}{16}$	1.54	$1\frac{15}{16}$	$1\frac{1}{8}$	↓	↓
×122	35.9	21.7	$21\frac{5}{8}$	0.600	$\frac{5}{8}$	$\frac{5}{16}$	12.4	$12\frac{3}{8}$	0.960	$\frac{15}{16}$	1.46	$1\frac{13}{16}$	$1\frac{1}{8}$	↓	↓
×111	32.7	21.5	$21\frac{1}{2}$	0.550	$\frac{9}{16}$	$\frac{5}{16}$	12.3	$12\frac{3}{8}$	0.875	$\frac{7}{8}$	1.38	$1\frac{3}{4}$	$1\frac{1}{8}$	↓	↓
×101^{c}	29.8	21.4	$21\frac{3}{8}$	0.500	$\frac{1}{2}$	$\frac{1}{4}$	12.3	$12\frac{1}{4}$	0.800	$\frac{13}{16}$	1.30	$1\frac{11}{16}$	$1\frac{1}{16}$	↓	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

Table 1-1 (continued)
W Shapes
Properties

W24 - W21

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
370	2.51	14.2	13400	957	11.1	1130	1160	170	3.27	267	3.92	25.3	201	186000
335	2.73	15.6	11900	864	11.0	1020	1030	152	3.23	238	3.86	25.0	152	161000
306	2.94	17.1	10700	789	10.9	922	919	137	3.20	214	3.81	24.9	117	142000
279	3.18	18.6	9600	718	10.8	835	823	124	3.17	193	3.76	24.6	90.5	125000
250	3.49	20.7	8490	644	10.7	744	724	110	3.14	171	3.71	24.5	66.6	108000
229	3.79	22.5	7650	588	10.7	675	651	99.4	3.11	154	3.67	24.3	51.3	96100
207	4.14	24.8	6820	531	10.6	606	578	88.8	3.08	137	3.62	24.1	38.3	84100
192	4.43	26.6	6260	491	10.5	559	530	81.8	3.07	126	3.60	24.0	30.8	76300
176	4.81	28.7	5680	450	10.5	511	479	74.3	3.04	115	3.57	23.9	23.9	68400
162	5.31	30.6	5170	414	10.4	468	443	68.4	3.05	105	3.57	23.8	18.5	62600
146	5.92	33.2	4580	371	10.3	418	391	60.5	3.01	93.2	3.53	23.7	13.4	54600
131	6.70	35.6	4020	329	10.2	370	340	53.0	2.97	81.5	3.49	23.5	9.50	47100
117	7.53	39.2	3540	291	10.1	327	297	46.5	2.94	71.4	3.46	23.4	6.72	40800
104	8.50	43.1	3100	258	10.1	289	259	40.7	2.91	62.4	3.42	23.3	4.72	35200
103	4.59	39.2	3000	245	10.0	280	119	26.5	1.99	41.5	2.40	23.6	7.07	16600
94	5.18	41.9	2700	222	9.87	254	109	24.0	1.98	37.5	2.40	23.4	5.26	15000
84	5.86	45.9	2370	196	9.79	224	94.4	20.9	1.95	32.6	2.37	23.3	3.70	12800
76	6.61	49.0	2100	176	9.69	200	82.5	18.4	1.92	28.6	2.34	23.2	2.68	11100
68	7.66	52.0	1830	154	9.55	177	70.4	15.7	1.87	24.5	2.30	23.1	1.87	9430
62	5.97	50.1	1550	131	9.23	153	34.5	9.80	1.38	15.7	1.75	23.2	1.71	4620
55	6.94	54.6	1350	114	9.11	134	29.1	8.30	1.34	13.3	1.71	23.1	1.18	3870
201	3.86	20.6	5310	461	9.47	530	542	86.1	3.02	133	3.55	21.4	40.9	62000
182	4.22	22.6	4730	417	9.40	476	483	77.2	3.00	119	3.51	21.2	30.7	54400
166	4.57	25.0	4280	380	9.36	432	435	70.0	2.99	108	3.48	21.1	23.6	48500
147	5.44	26.1	3630	329	9.17	373	376	60.1	2.95	92.6	3.45	20.9	15.4	41100
132	6.01	28.9	3220	295	9.12	333	333	53.5	2.93	82.3	3.42	20.8	11.3	36000
122	6.45	31.3	2960	273	9.09	307	305	49.2	2.92	75.6	3.40	20.7	8.98	32700
111	7.05	34.1	2670	249	9.05	279	274	44.5	2.90	68.2	3.37	20.6	6.83	29200
101	7.68	37.5	2420	227	9.02	253	248	40.3	2.89	61.7	3.35	20.6	5.21	26200

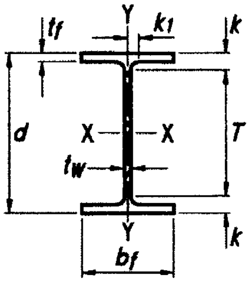


Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W21×93	27.3	21.6	21 5/8	0.580	9/16	5/16	8.42	8 3/8	0.930	15/16	1.43	1 5/8	15/16	18 3/8	5 1/2
×83[c]	24.3	21.4	21 3/8	0.515	1/2	1/4	8.36	8 3/8	0.835	13/16	1.34	1 1/2	7/8	↓	↓
×73[c]	21.5	21.2	21 1/4	0.455	7/16	1/4	8.30	8 1/4	0.740	3/4	1.24	1 7/16	7/8	↓	↓
×68[c]	20.0	21.1	21 1/8	0.430	7/16	1/4	8.27	8 1/4	0.685	11/16	1.19	1 3/8	7/8	↓	↓
×62[c]	18.3	21.0	21	0.400	3/8	3/16	8.24	8 1/4	0.615	5/8	1.12	1 5/16	13/16	↓	↓
×55[c]	16.2	20.8	20 3/4	0.375	3/8	3/16	8.22	8 1/4	0.522	1/2	1.02	1 3/16	13/16	↓	↓
×48[c,f]	14.1	20.6	20 5/8	0.350	3/8	3/16	8.14	8 1/8	0.430	7/16	0.930	1 1/8	13/16	↓	↓
W21×57[c]	16.7	21.1	21	0.405	3/8	3/16	6.56	6 1/2	0.650	5/8	1.15	1 5/16	13/16	18 3/8	3 1/2
×50[c]	14.7	20.8	20 7/8	0.380	3/8	3/16	6.53	6 1/2	0.535	9/16	1.04	1 1/4	13/16	↓	↓
×44[c]	13.0	20.7	20 5/8	0.350	3/8	3/16	6.50	6 1/2	0.450	7/16	0.950	1 1/8	13/16	↓	↓
W18×311[h]	91.6	22.3	22 3/8	1.52	1 1/2	3/4	12.0	12	2.74	2 3/4	3.24	3 7/16	1 3/8	15 1/2	5 1/2
×283[h]	83.3	21.9	21 7/8	1.40	1 3/8	11/16	11.9	11 7/8	2.50	2 1/2	3.00	3 3/16	1 5/16	↓	↓
×258[h]	75.9	21.5	21 1/2	1.28	1 1/4	5/8	11.8	11 3/4	2.30	2 5/16	2.70	3	1 1/4	↓	↓
×234[h]	68.8	21.1	21	1.16	1 3/16	5/8	11.7	11 5/8	2.11	2 1/8	2.51	2 3/4	1 3/16	↓	↓
×211	62.1	20.7	20 5/8	1.06	1 1/16	9/16	11.6	11 1/2	1.91	1 15/16	2.31	2 9/16	1 3/16	↓	↓
×192	56.4	20.4	20 3/8	0.960	15/16	1/2	11.5	11 1/2	1.75	1 3/4	2.15	2 7/16	1 1/8	↓	↓
×175	51.3	20.0	20	0.890	7/8	7/16	11.4	11 3/8	1.59	1 9/16	1.99	2 7/16	1 1/4	15 1/8	↓
×158	46.3	19.7	19 3/4	0.810	13/16	7/16	11.3	11 1/4	1.44	1 7/16	1.84	2 3/8	1 1/4	↓	↓
×143	42.1	19.5	19 1/2	0.730	3/4	3/8	11.2	11 1/4	1.32	1 5/16	1.72	2 3/16	1 3/16	↓	↓
×130	38.2	19.3	19 1/4	0.670	11/16	3/8	11.2	11 1/8	1.20	1 3/16	1.60	2 1/16	1 3/16	↓	↓
×119	35.1	19.0	19	0.655	5/8	5/16	11.3	11 1/4	1.06	1 1/16	1.46	1 15/16	1 3/16	↓	↓
×106	31.1	18.7	18 3/4	0.590	9/16	5/16	11.2	11 1/4	0.940	15/16	1.34	1 13/16	1 1/8	↓	↓
×97	28.5	18.6	18 5/8	0.535	9/16	5/16	11.1	11 1/8	0.870	7/8	1.27	1 3/4	1 1/8	↓	↓
×86	25.3	18.4	18 3/8	0.480	1/2	1/4	11.1	11 1/8	0.770	3/4	1.17	1 5/8	1 1/16	↓	↓
×76[c]	22.3	18.2	18 1/4	0.425	7/16	1/4	11.0	11	0.680	11/16	1.08	1 9/16	1 1/16	↓	↓
W18×71	20.8	18.5	18 1/2	0.495	1/2	1/4	7.64	7 5/8	0.810	13/16	1.21	1 1/2	7/8	15 1/2	3 1/2[g]
×65	19.1	18.4	18 3/8	0.450	7/16	1/4	7.59	7 5/8	0.750	3/4	1.15	1 7/16	7/8	↓	↓
×60[c]	17.6	18.2	18 1/4	0.415	7/16	1/4	7.56	7 1/2	0.695	11/16	1.10	1 3/8	13/16	↓	↓
×55[c]	16.2	18.1	18 1/8	0.390	3/8	3/16	7.53	7 1/2	0.630	5/8	1.03	1 5/16	13/16	↓	↓
×50[c]	14.7	18.0	18	0.355	3/8	3/16	7.50	7 1/2	0.570	9/16	0.972	1 1/4	13/16	↓	↓
W18×46[c]	13.5	18.1	18	0.360	3/8	3/16	6.06	6	0.605	5/8	1.01	1 1/4	13/16	15 1/2	3 1/2[g]
×40[c]	11.8	17.9	17 7/8	0.315	5/16	3/16	6.02	6	0.525	1/2	0.927	1 3/16	13/16	↓	↓
×35[c]	10.3	17.7	17 3/4	0.300	5/16	3/16	6.00	6	0.425	7/16	0.827	1 1/8	3/4	↓	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Table 1-1 (continued)
W Shapes
Properties

W21 - W18

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
93	4.53	32.3	2070	192	8.70	221	92.9	22.1	1.84	34.7	2.24	20.7	6.03	9940
83	5.00	36.4	1830	171	8.67	196	81.4	19.5	1.83	30.5	2.21	20.6	4.34	8630
73	5.60	41.2	1600	151	8.64	172	70.6	17.0	1.81	26.6	2.19	20.5	3.02	7410
68	6.04	43.6	1480	140	8.60	160	64.7	15.7	1.80	24.4	2.17	20.4	2.45	6760
62	6.70	46.9	1330	127	8.54	144	57.5	14.0	1.77	21.7	2.15	20.4	1.83	5960
55	7.87	50.0	1140	110	8.40	126	48.4	11.8	1.73	18.4	2.11	20.3	1.24	4980
48	9.47	53.6	959	93.0	8.24	107	38.7	9.52	1.66	14.9	2.05	20.2	0.803	3950
57	5.04	46.3	1170	111	8.36	129	30.6	9.35	1.35	14.8	1.68	20.4	1.77	3190
50	6.10	49.4	984	94.5	8.18	110	24.9	7.64	1.30	12.2	1.64	20.3	1.14	2570
44	7.22	53.6	843	81.6	8.06	95.4	20.7	6.37	1.26	10.2	1.60	20.2	0.770	2110
311	2.19	10.4	6970	624	8.72	754	795	132	2.95	207	3.53	19.6	176	76200
283	2.38	11.3	6170	565	8.61	676	704	118	2.91	185	3.47	19.4	134	65900
258	2.56	12.5	5510	514	8.53	611	628	107	2.88	166	3.42	19.2	103	57600
234	2.76	13.8	4900	466	8.44	549	558	95.8	2.85	149	3.37	19.0	78.7	50100
211	3.02	15.1	4330	419	8.35	490	493	85.3	2.82	132	3.32	18.8	58.6	43400
192	3.27	16.7	3870	380	8.28	442	440	76.8	2.79	119	3.28	18.6	44.7	38000
175	3.58	18.0	3450	344	8.20	398	391	68.8	2.76	106	3.24	18.5	33.8	33300
158	3.92	19.8	3060	310	8.12	356	347	61.4	2.74	94.8	3.20	18.3	25.2	29000
143	4.25	22.0	2750	282	8.09	322	311	55.5	2.72	85.4	3.17	18.2	19.2	25700
130	4.65	23.9	2460	256	8.03	290	278	49.9	2.70	76.7	3.13	18.1	14.5	22700
119	5.31	24.5	2190	231	7.90	262	253	44.9	2.69	69.1	3.13	17.9	10.6	20300
106	5.96	27.2	1910	204	7.84	230	220	39.4	2.66	60.5	3.10	17.8	7.48	17400
97	6.41	30.0	1750	188	7.82	211	201	36.1	2.65	55.3	3.08	17.7	5.86	15800
86	7.20	33.4	1530	166	7.77	186	175	31.6	2.63	48.4	3.05	17.6	4.10	13600
76	8.11	37.8	1330	146	7.73	163	152	27.6	2.61	42.2	3.02	17.5	2.83	11700
71	4.71	32.4	1170	127	7.50	146	60.3	15.8	1.70	24.7	2.05	17.7	3.49	4700
65	5.06	35.7	1070	117	7.49	133	54.8	14.4	1.69	22.5	2.03	17.6	2.73	4240
60	5.44	38.7	984	108	7.47	123	50.1	13.3	1.68	20.6	2.02	17.5	2.17	3850
55	5.98	41.1	890	98.3	7.41	112	44.9	11.9	1.67	18.5	2.00	17.5	1.66	3430
50	6.57	45.2	800	88.9	7.38	101	40.1	10.7	1.65	16.6	1.98	17.4	1.24	3040
46	5.01	44.6	712	78.8	7.25	90.7	22.5	7.43	1.29	11.7	1.58	17.5	1.22	1720
40	5.73	50.9	612	68.4	7.21	78.4	19.1	6.35	1.27	10.0	1.56	17.4	0.810	1440
35	7.06	53.5	510	57.6	7.04	66.5	15.3	5.12	1.22	8.06	1.52	17.3	0.506	1140

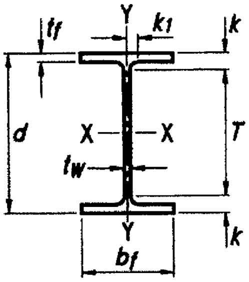


Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W16×100	29.5	17.0	17	0.585	9/16	5/16	10.4	10 3/8	0.985	1	1.39	1 7/8	1 1/8	13 1/4	5 1/2
×89	26.2	16.8	16 3/4	0.525	1/2	1/4	10.4	10 3/8	0.875	7/8	1.28	1 3/4	1 1/16		
×77	22.6	16.5	16 1/2	0.455	7/16	1/4	10.3	10 1/4	0.760	3/4	1.16	1 5/8	1 1/16		
×67^{c}	19.7	16.3	16 3/8	0.395	3/8	3/16	10.2	10 1/4	0.665	11/16	1.07	1 9/16	1	↓	↓
W16×57	16.8	16.4	16 3/8	0.430	7/16	1/4	7.12	7 1/8	0.715	11/16	1.12	1 3/8	7/8	13 5/8	3 1/2^{g}
×50^{c}	14.7	16.3	16 1/4	0.380	3/8	3/16	7.07	7 1/8	0.630	5/8	1.03	1 5/16	13/16		
×45^{c}	13.3	16.1	16 1/8	0.345	3/8	3/16	7.04	7	0.565	9/16	0.967	1 1/4	13/16		
×40^{c}	11.8	16.0	16	0.305	5/16	3/16	7.00	7	0.505	1/2	0.907	1 3/16	13/16		
×36^{c}	10.6	15.9	15 7/8	0.295	5/16	3/16	6.99	7	0.430	7/16	0.832	1 1/8	3/4	↓	↓
W16×31^{c}	9.13	15.9	15 7/8	0.275	1/4	1/8	5.53	5 1/2	0.440	7/16	0.842	1 1/8	3/4	13 5/8	3 1/2
×26^{c,v}	7.68	15.7	15 3/4	0.250	1/4	1/8	5.50	5 1/2	0.345	3/8	0.747	1 1/16	3/4	13 5/8	3 1/2
W14×730^{h}	215	22.4	22 3/8	3.07	3 1/16	1 9/16	17.9	17 7/8	4.91	4 15/16	5.51	6 3/16	2 3/4	10	3-7 1/2-3^{g}
×665^{h}	196	21.6	21 5/8	2.83	2 13/16	1 7/16	17.7	17 5/8	4.52	4 1/2	5.12	5 13/16	2 5/8		3-7 1/2-3^{g}
×605^{h}	178	20.9	20 7/8	2.60	2 5/8	1 5/16	17.4	17 3/8	4.16	4 3/16	4.76	5 7/16	2 1/2		3-7 1/2-3
×550^{h}	162	20.2	20 1/4	2.38	2 3/8	1 3/16	17.2	17 1/4	3.82	3 13/16	4.42	5 1/8	2 3/8		
×500^{h}	147	19.6	19 5/8	2.19	2 3/16	1 1/8	17.0	17	3.50	3 1/2	4.10	4 13/16	2 5/16		
×455^{h}	134	19.0	19	2.02	2	1	16.8	16 7/8	3.21	3 3/16	3.81	4 1/2	2 1/4		
×426^{h}	125	18.7	18 5/8	1.88	1 7/8	15/16	16.7	16 3/4	3.04	3 1/16	3.63	4 5/16	2 1/8		
×398^{h}	117	18.3	18 1/4	1.77	1 3/4	7/8	16.6	16 5/8	2.85	2 7/8	3.44	4 1/8	2 1/8		
×370^{h}	109	17.9	17 7/8	1.66	1 5/8	13/16	16.5	16 1/2	2.66	2 11/16	3.26	3 15/16	2 1/16		
×342^{h}	101	17.5	17 1/2	1.54	1 9/16	13/16	16.4	16 3/8	2.47	2 1/2	3.07	3 3/4	2		
×311^{h}	91.4	17.1	17 1/8	1.41	1 7/16	3/4	16.2	16 1/4	2.26	2 1/4	2.86	3 9/16	1 15/16		
×283^{h}	83.3	16.7	16 3/4	1.29	1 5/16	11/16	16.1	16 1/8	2.07	2 1/16	2.67	3 3/8	1 7/8		
×257	75.6	16.4	16 3/8	1.18	1 3/16	5/8	16.0	16	1.89	1 7/8	2.49	3 3/16	1 13/16		
×233	68.5	16.0	16	1.07	1 1/16	9/16	15.9	15 7/8	1.72	1 3/4	2.32	3	1 3/4		
×211	62.0	15.7	15 3/4	0.980	1	1/2	15.8	15 3/4	1.56	1 9/16	2.16	2 7/8	1 11/16		
×193	56.8	15.5	15 1/2	0.890	7/8	7/16	15.7	15 3/4	1.44	1 7/16	2.04	2 3/4	1 11/16		
×176	51.8	15.2	15 1/4	0.830	13/16	7/16	15.7	15 5/8	1.31	1 5/16	1.91	2 5/8	1 5/8		
×159	46.7	15.0	15	0.745	3/4	3/8	15.6	15 5/8	1.19	1 3/16	1.79	2 1/2	1 9/16		
×145	42.7	14.8	14 3/4	0.680	11/16	3/8	15.5	15 1/2	1.09	1 1/16	1.69	2 3/8	1 9/16	↓	↓

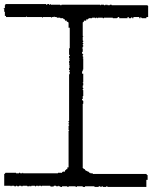
[c] Shape is slender for compression with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi.

Table 1–1 (continued)
W Shapes
Properties



W16 – W14

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
100	5.29	24.3	1490	175	7.10	198	186	35.7	2.51	54.9	2.92	16.0	7.73	11900
89	5.92	27.0	1300	155	7.05	175	163	31.4	2.49	48.1	2.88	15.9	5.45	10200
77	6.77	31.2	1110	134	7.00	150	138	26.9	2.47	41.1	2.85	15.8	3.57	8590
67	7.70	35.9	954	117	6.96	130	119	23.2	2.46	35.5	2.82	15.7	2.39	7300
57	4.98	33.0	758	92.2	6.72	105	43.1	12.1	1.60	18.9	1.92	15.7	2.22	2660
50	5.61	37.4	659	81.0	6.68	92.0	37.2	10.5	1.59	16.3	1.89	15.6	1.52	2270
45	6.23	41.1	586	72.7	6.65	82.3	32.8	9.34	1.57	14.5	1.88	15.6	1.11	1990
40	6.93	46.5	518	64.7	6.63	73.0	28.9	8.25	1.57	12.7	1.86	15.5	0.794	1730
36	8.12	48.1	448	56.5	6.51	64.0	24.5	7.00	1.52	10.8	1.83	15.4	0.545	1460
31	6.28	51.6	375	47.2	6.41	54.0	12.4	4.49	1.17	7.03	1.42	15.4	0.461	739
26	7.97	56.8	301	38.4	6.26	44.2	9.59	3.49	1.12	5.48	1.38	15.3	0.262	565
730	1.82	3.71	14300	1280	8.17	1660	4720	527	4.69	816	5.68	17.5	1450	362000
665	1.95	4.03	12400	1150	7.98	1480	4170	472	4.62	730	5.57	17.1	1120	305000
605	2.09	4.39	10800	1040	7.80	1320	3680	423	4.55	652	5.46	16.8	869	258000
550	2.25	4.79	9430	931	7.63	1180	3250	378	4.49	583	5.36	16.4	669	219000
500	2.43	5.21	8210	838	7.48	1050	2880	339	4.43	522	5.26	16.1	514	187000
455	2.62	5.66	7190	756	7.33	936	2560	304	4.38	468	5.17	15.8	395	160000
426	2.75	6.08	6600	706	7.26	869	2360	283	4.34	434	5.11	15.6	331	144000
398	2.92	6.44	6000	656	7.16	801	2170	262	4.31	402	5.06	15.4	273	129000
370	3.10	6.89	5440	607	7.07	736	1990	241	4.27	370	5.00	15.3	222	116000
342	3.31	7.41	4900	558	6.98	672	1810	221	4.24	338	4.94	15.1	178	103000
311	3.59	8.09	4330	506	6.88	603	1610	199	4.20	304	4.87	14.9	136	89100
283	3.89	8.84	3840	459	6.79	542	1440	179	4.17	274	4.81	14.7	104	77700
257	4.23	9.71	3400	415	6.71	487	1290	161	4.13	246	4.75	14.5	79.1	67800
233	4.62	10.7	3010	375	6.63	436	1150	145	4.10	221	4.69	14.3	59.5	59000
211	5.06	11.6	2660	338	6.55	390	1030	130	4.07	198	4.64	14.2	44.6	51500
193	5.45	12.8	2400	310	6.50	355	931	119	4.05	180	4.59	14.0	34.8	45900
176	5.97	13.7	2140	281	6.43	320	838	107	4.02	163	4.55	13.9	26.5	40500
159	6.54	15.3	1900	254	6.38	287	748	96.2	4.00	146	4.51	13.8	19.7	35600
145	7.11	16.8	1710	232	6.33	260	677	87.3	3.98	133	4.47	13.7	15.2	31700

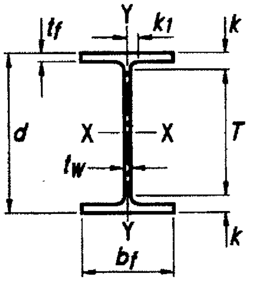


Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W14×132	38.8	14.7	$14\frac{5}{8}$	0.645	$\frac{5}{8}$	$\frac{5}{16}$	14.7	$14\frac{3}{4}$	1.03	1	1.63	$2\frac{5}{16}$	$1\frac{9}{16}$	10	$5\frac{1}{2}$
×120	35.3	14.5	$14\frac{1}{2}$	0.590	$\frac{9}{16}$	$\frac{5}{16}$	14.7	$14\frac{5}{8}$	0.940	$\frac{15}{16}$	1.54	$2\frac{1}{4}$	$1\frac{1}{2}$	↓	↓
×109	32.0	14.3	$14\frac{3}{8}$	0.525	$\frac{1}{2}$	$\frac{1}{4}$	14.6	$14\frac{5}{8}$	0.860	$\frac{7}{8}$	1.46	$2\frac{3}{16}$	$1\frac{1}{2}$	↓	↓
×99[f]	29.1	14.2	$14\frac{1}{8}$	0.485	$\frac{1}{2}$	$\frac{1}{4}$	14.6	$14\frac{5}{8}$	0.780	$\frac{3}{4}$	1.38	$2\frac{1}{16}$	$1\frac{7}{16}$	↓	↓
×90[f]	26.5	14.0	14	0.440	$\frac{7}{16}$	$\frac{1}{4}$	14.5	$14\frac{1}{2}$	0.710	$\frac{11}{16}$	1.31	2	$1\frac{7}{16}$	↓	↓
W14×82	24.0	14.3	$14\frac{1}{4}$	0.510	$\frac{1}{2}$	$\frac{1}{4}$	10.1	$10\frac{1}{8}$	0.855	$\frac{7}{8}$	1.45	$1\frac{11}{16}$	$1\frac{1}{16}$	$10\frac{7}{8}$	$5\frac{1}{2}$
×74	21.8	14.2	$14\frac{1}{8}$	0.450	$\frac{7}{16}$	$\frac{1}{4}$	10.1	$10\frac{1}{8}$	0.785	$\frac{13}{16}$	1.38	$1\frac{5}{8}$	$1\frac{1}{16}$	↓	↓
×68	20.0	14.0	14	0.415	$\frac{7}{16}$	$\frac{1}{4}$	10.0	10	0.720	$\frac{3}{4}$	1.31	$1\frac{9}{16}$	$1\frac{1}{16}$	↓	↓
×61	17.9	13.9	$13\frac{7}{8}$	0.375	$\frac{3}{8}$	$\frac{3}{16}$	10.0	10	0.645	$\frac{5}{8}$	1.24	$1\frac{1}{2}$	1	↓	↓
W14×53	15.6	13.9	$13\frac{7}{8}$	0.370	$\frac{3}{8}$	$\frac{3}{16}$	8.06	8	0.660	$\frac{11}{16}$	1.25	$1\frac{1}{2}$	1	$10\frac{7}{8}$	$5\frac{1}{2}$
×48	14.1	13.8	$13\frac{3}{4}$	0.340	$\frac{5}{16}$	$\frac{3}{16}$	8.03	8	0.595	$\frac{5}{8}$	1.19	$1\frac{7}{16}$	1	↓	↓
×43[c]	12.6	13.7	$13\frac{5}{8}$	0.305	$\frac{5}{16}$	$\frac{3}{16}$	8.00	8	0.530	$\frac{1}{2}$	1.12	$1\frac{3}{8}$	1	↓	↓
W14×38[c]	11.2	14.1	$14\frac{1}{8}$	0.310	$\frac{5}{16}$	$\frac{3}{16}$	6.77	$6\frac{3}{4}$	0.515	$\frac{1}{2}$	0.915	$1\frac{1}{4}$	$\frac{13}{16}$	$11\frac{5}{8}$	$3\frac{1}{2}$[g]
×34[c]	10.0	14.0	14	0.285	$\frac{5}{16}$	$\frac{3}{16}$	6.75	$6\frac{3}{4}$	0.455	$\frac{7}{16}$	0.855	$1\frac{3}{16}$	$\frac{3}{4}$	↓	$3\frac{1}{2}$
×30[c]	8.85	13.8	$13\frac{7}{8}$	0.270	$\frac{1}{4}$	$\frac{1}{8}$	6.73	$6\frac{3}{4}$	0.385	$\frac{3}{8}$	0.785	$1\frac{1}{8}$	$\frac{3}{4}$	↓	$3\frac{1}{2}$
W14×26[c]	7.69	13.9	$13\frac{7}{8}$	0.255	$\frac{1}{4}$	$\frac{1}{8}$	5.03	5	0.420	$\frac{7}{16}$	0.820	$1\frac{1}{8}$	$\frac{3}{4}$	$11\frac{5}{8}$	$2\frac{3}{4}$[g]
×22[c]	6.49	13.7	$13\frac{3}{4}$	0.230	$\frac{1}{4}$	$\frac{1}{8}$	5.00	5	0.335	$\frac{5}{16}$	0.735	$1\frac{1}{16}$	$\frac{3}{4}$	$11\frac{5}{8}$	$2\frac{3}{4}$[g]
W12×336[h]	98.8	16.8	$16\frac{7}{8}$	1.78	$1\frac{3}{4}$	$\frac{7}{8}$	13.4	$13\frac{3}{8}$	2.96	$2\frac{15}{16}$	3.55	$3\frac{7}{8}$	$1\frac{11}{16}$	$9\frac{1}{8}$	$5\frac{1}{2}$
×305[h]	89.6	16.3	$16\frac{3}{8}$	1.63	$1\frac{5}{8}$	$\frac{13}{16}$	13.2	$13\frac{1}{4}$	2.71	$2\frac{11}{16}$	3.30	$3\frac{5}{8}$	$1\frac{5}{8}$	↓	↓
×279[h]	81.9	15.9	$15\frac{7}{8}$	1.53	$1\frac{1}{2}$	$\frac{3}{4}$	13.1	$13\frac{1}{8}$	2.47	$2\frac{1}{2}$	3.07	$3\frac{3}{8}$	$1\frac{5}{8}$	↓	↓
×252[h]	74.0	15.4	$15\frac{3}{8}$	1.40	$1\frac{3}{8}$	$\frac{11}{16}$	13.0	13	2.25	$2\frac{1}{4}$	2.85	$3\frac{1}{8}$	$1\frac{1}{2}$	↓	↓
×230[h]	67.7	15.1	15	1.29	$1\frac{5}{16}$	$\frac{11}{16}$	12.9	$12\frac{7}{8}$	2.07	$2\frac{1}{16}$	2.67	$2\frac{15}{16}$	$1\frac{1}{2}$	↓	↓
×210	61.8	14.7	$14\frac{3}{4}$	1.18	$1\frac{3}{16}$	$\frac{5}{8}$	12.8	$12\frac{3}{4}$	1.90	$1\frac{7}{8}$	2.50	$2\frac{13}{16}$	$1\frac{7}{16}$	↓	↓
×190	55.8	14.4	$14\frac{3}{8}$	1.06	$1\frac{1}{16}$	$\frac{9}{16}$	12.7	$12\frac{5}{8}$	1.74	$1\frac{3}{4}$	2.33	$2\frac{5}{8}$	$1\frac{3}{8}$	↓	↓
×170	50.0	14.0	14	0.960	$\frac{15}{16}$	$\frac{1}{2}$	12.6	$12\frac{5}{8}$	1.56	$1\frac{9}{16}$	2.16	$2\frac{7}{16}$	$1\frac{5}{16}$	↓	↓
×152	44.7	13.7	$13\frac{3}{4}$	0.870	$\frac{7}{8}$	$\frac{7}{16}$	12.5	$12\frac{1}{2}$	1.40	$1\frac{3}{8}$	2.00	$2\frac{5}{16}$	$1\frac{1}{4}$	↓	↓
×136	39.9	13.4	$13\frac{3}{8}$	0.790	$\frac{13}{16}$	$\frac{7}{16}$	12.4	$12\frac{3}{8}$	1.25	$1\frac{1}{4}$	1.85	$2\frac{1}{8}$	$1\frac{1}{4}$	↓	↓
×120	35.3	13.1	$13\frac{1}{8}$	0.710	$\frac{11}{16}$	$\frac{3}{8}$	12.3	$12\frac{3}{8}$	1.11	$1\frac{1}{8}$	1.70	2	$1\frac{3}{16}$	↓	↓
×106	31.2	12.9	$12\frac{7}{8}$	0.610	$\frac{5}{8}$	$\frac{5}{16}$	12.2	$12\frac{1}{4}$	0.990	1	1.59	$1\frac{7}{8}$	$1\frac{1}{8}$	↓	↓
×96	28.2	12.7	$12\frac{3}{4}$	0.550	$\frac{9}{16}$	$\frac{5}{16}$	12.2	$12\frac{1}{8}$	0.900	$\frac{7}{8}$	1.50	$1\frac{13}{16}$	$1\frac{1}{8}$	↓	↓
×87	25.6	12.5	$12\frac{1}{2}$	0.515	$\frac{1}{2}$	$\frac{1}{4}$	12.1	$12\frac{1}{8}$	0.810	$\frac{13}{16}$	1.41	$1\frac{11}{16}$	$1\frac{1}{16}$	↓	↓
×79	23.2	12.4	$12\frac{3}{8}$	0.470	$\frac{1}{2}$	$\frac{1}{4}$	12.1	$12\frac{1}{8}$	0.735	$\frac{3}{4}$	1.33	$1\frac{5}{8}$	$1\frac{1}{16}$	↓	↓
×72	21.1	12.3	$12\frac{1}{4}$	0.430	$\frac{7}{16}$	$\frac{1}{4}$	12.0	12	0.670	$\frac{11}{16}$	1.27	$1\frac{9}{16}$	$1\frac{1}{16}$	↓	↓
×65[f]	19.1	12.1	$12\frac{1}{8}$	0.390	$\frac{3}{8}$	$\frac{3}{16}$	12.0	12	0.605	$\frac{5}{8}$	1.20	$1\frac{1}{2}$	1	↓	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Table 1-1 (continued)
W Shapes
Properties

W14 – W12

Nominal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
132	7.15	17.7	1530	209	6.28	234	548	74.5	3.76	113	4.23	13.6	12.3	25500
120	7.80	19.3	1380	190	6.24	212	495	67.5	3.74	102	4.20	13.5	9.37	22700
109	8.49	21.7	1240	173	6.22	192	447	61.2	3.73	92.7	4.17	13.5	7.12	20200
99	9.34	23.5	1110	157	6.17	173	402	55.2	3.71	83.6	4.14	13.4	5.37	18000
90	10.2	25.9	999	143	6.14	157	362	49.9	3.70	75.6	4.11	13.3	4.06	16000
82	5.92	22.4	881	123	6.05	139	148	29.3	2.48	44.8	2.85	13.5	5.07	6710
74	6.41	25.4	795	112	6.04	126	134	26.6	2.48	40.5	2.82	13.4	3.87	5990
68	6.97	27.5	722	103	6.01	115	121	24.2	2.46	36.9	2.80	13.3	3.01	5380
61	7.75	30.4	640	92.1	5.98	102	107	21.5	2.45	32.8	2.78	13.2	2.19	4710
53	6.11	30.9	541	77.8	5.89	87.1	57.7	14.3	1.92	22.0	2.22	13.3	1.94	2540
48	6.75	33.6	484	70.2	5.85	78.4	51.4	12.8	1.91	19.6	2.20	13.2	1.45	2240
43	7.54	37.4	428	62.6	5.82	69.6	45.2	11.3	1.89	17.3	2.18	13.1	1.05	1950
38	6.57	39.6	385	54.6	5.87	61.5	26.7	7.88	1.55	12.1	1.82	13.6	0.798	1230
34	7.41	43.1	340	48.6	5.83	54.6	23.3	6.91	1.53	10.6	1.80	13.5	0.569	1070
30	8.74	45.4	291	42.0	5.73	47.3	19.6	5.82	1.49	8.99	1.77	13.5	0.380	887
26	5.98	48.1	245	35.3	5.65	40.2	8.91	3.55	1.08	5.54	1.31	13.5	0.358	405
22	7.46	53.3	199	29.0	5.54	33.2	7.00	2.80	1.04	4.39	1.27	13.4	0.208	314
336	2.26	5.47	4060	483	6.41	603	1190	177	3.47	274	4.13	13.9	243	57000
305	2.45	5.98	3550	435	6.29	537	1050	159	3.42	244	4.05	13.6	185	48600
279	2.66	6.35	3110	393	6.16	481	937	143	3.38	220	4.00	13.4	143	42000
252	2.89	6.96	2720	353	6.06	428	828	127	3.34	196	3.93	13.2	108	35800
230	3.11	7.56	2420	321	5.97	386	742	115	3.31	177	3.87	13.0	83.8	31200
210	3.37	8.23	2140	292	5.89	348	664	104	3.28	159	3.82	12.8	64.7	27200
190	3.65	9.16	1890	263	5.82	311	589	93.0	3.25	143	3.76	12.6	48.8	23600
170	4.03	10.1	1650	235	5.74	275	517	82.3	3.22	126	3.71	12.5	35.6	20100
152	4.46	11.2	1430	209	5.66	243	454	72.8	3.19	111	3.66	12.3	25.8	17200
136	4.96	12.3	1240	186	5.58	214	398	64.2	3.16	98.0	3.61	12.2	18.5	14700
120	5.57	13.7	1070	163	5.51	186	345	56.0	3.13	85.4	3.56	12.0	12.9	12400
106	6.17	15.9	933	145	5.47	164	301	49.3	3.11	75.1	3.52	11.9	9.13	10700
96	6.76	17.7	833	131	5.44	147	270	44.4	3.09	67.5	3.49	11.8	6.85	9410
87	7.48	18.9	740	118	5.38	132	241	39.7	3.07	60.4	3.46	11.7	5.10	8270
79	8.22	20.7	662	107	5.34	119	216	35.8	3.05	54.3	3.43	11.6	3.84	7330
72	8.99	22.6	597	97.4	5.31	108	195	32.4	3.04	49.2	3.40	11.6	2.93	6540
65	9.92	24.9	533	87.9	5.28	96.8	174	29.1	3.02	44.1	3.38	11.5	2.18	5780

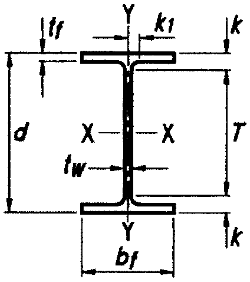


Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W12×58	17.0	12.2	12 1/4	0.360	3/8	3/16	10.0	10	0.640	5/8	1.24	1 1/2	15/16	9 1/4	5 1/2
×53	15.6	12.1	12	0.345	3/8	3/16	10.0	10	0.575	9/16	1.18	1 3/8	15/16	9 1/4	5 1/2
W12×50	14.6	12.2	12 1/4	0.370	3/8	3/16	8.08	8 1/8	0.640	5/8	1.14	1 1/2	15/16	9 1/4	5 1/2
×45	13.1	12.1	12	0.335	5/16	3/16	8.05	8	0.575	9/16	1.08	1 3/8	15/16	↓	↓
×40	11.7	11.9	12	0.295	5/16	3/16	8.01	8	0.515	1/2	1.02	1 3/8	7/8	↓	↓
W12×35[c]	10.3	12.5	12 1/2	0.300	5/16	3/16	6.56	6 1/2	0.520	1/2	0.820	1 3/16	3/4	10 1/8	3 1/2
×30[c]	8.79	12.3	12 3/8	0.260	1/4	1/8	6.52	6 1/2	0.440	7/16	0.740	1 1/8	3/4	↓	↓
×26[c]	7.65	12.2	12 1/4	0.230	1/4	1/8	6.49	6 1/2	0.380	3/8	0.680	1 1/16	3/4	↓	↓
W12×22[c]	6.48	12.3	12 1/4	0.260	1/4	1/8	4.03	4	0.425	7/16	0.725	15/16	5/8	10 3/8	2 1/4[g]
×19[c]	5.57	12.2	12 1/8	0.235	1/4	1/8	4.01	4	0.350	3/8	0.650	7/8	9/16	↓	↓
×16[c]	4.71	12.0	12	0.220	1/4	1/8	3.99	4	0.265	1/4	0.565	13/16	9/16	↓	↓
×14[c,v]	4.16	11.9	11 7/8	0.200	3/16	1/8	3.97	4	0.225	1/4	0.525	3/4	9/16	↓	↓
W10×112	32.9	11.4	11 3/8	0.755	3/4	3/8	10.4	10 3/8	1.25	1 1/4	1.75	1 15/16	1	7 1/2	5 1/2
×100	29.4	11.1	11 1/8	0.680	11/16	3/8	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	1	↓	↓
×88	25.9	10.8	10 7/8	0.605	5/8	5/16	10.3	10 1/4	0.990	1	1.49	1 11/16	15/16	↓	↓
×77	22.6	10.6	10 5/8	0.530	1/2	1/4	10.2	10 1/4	0.870	7/8	1.37	1 9/16	7/8	↓	↓
×68	20.0	10.4	10 3/8	0.470	1/2	1/4	10.1	10 1/8	0.770	3/4	1.27	1 7/16	7/8	↓	↓
×60	17.6	10.2	10 1/4	0.420	7/16	1/4	10.1	10 1/8	0.680	11/16	1.18	1 3/8	13/16	↓	↓
×54	15.8	10.1	10 1/8	0.370	3/8	3/16	10.0	10	0.615	5/8	1.12	1 5/16	13/16	↓	↓
×49	14.4	10.0	10	0.340	5/16	3/16	10.0	10	0.560	9/16	1.06	1 1/4	13/16	↓	↓
W10×45	13.3	10.1	10 1/8	0.350	3/8	3/16	8.02	8	0.620	5/8	1.12	1 5/16	13/16	7 1/2	5 1/2
×39	11.5	9.92	9 7/8	0.315	5/16	3/16	7.99	8	0.530	1/2	1.03	1 3/16	13/16	↓	↓
×33	9.71	9.73	9 3/4	0.290	5/16	3/16	7.96	8	0.435	7/16	0.935	1 1/8	3/4	↓	↓
W10×30	8.84	10.5	10 1/2	0.300	5/16	3/16	5.81	5 3/4	0.510	1/2	0.810	1 1/8	11/16	8 1/4	2 3/4[g]
×26	7.61	10.3	10 3/8	0.260	1/4	1/8	5.77	5 3/4	0.440	7/16	0.740	1 1/16	11/16	↓	↓
×22[c]	6.49	10.2	10 1/8	0.240	1/4	1/8	5.75	5 3/4	0.360	3/8	0.660	15/16	5/8	↓	↓
W10×19	5.62	10.2	10 1/4	0.250	1/4	1/8	4.02	4	0.395	3/8	0.695	15/16	5/8	8 3/8	2 1/4[g]
×17[c]	4.99	10.1	10 1/8	0.240	1/4	1/8	4.01	4	0.330	5/16	0.630	7/8	9/16	↓	↓
×15[c]	4.41	10.0	10	0.230	1/4	1/8	4.00	4	0.270	1/4	0.570	13/16	9/16	↓	↓
×12[c,f]	3.54	9.87	9 7/8	0.190	3/16	1/8	3.96	4	0.210	3/16	0.510	3/4	9/16	↓	↓

[c] Shape is slender for compression with F_y = 50 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi.

Table 1-1 (continued)
W Shapes
Properties

W12 – W10

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
58	7.82	27.0	475	78.0	5.28	86.4	107	21.4	2.51	32.5	2.82	11.6	2.10	3570
53	8.69	28.1	425	70.6	5.23	77.9	95.8	19.2	2.48	29.1	2.79	11.5	1.58	3160
50	6.31	26.8	391	64.2	5.18	71.9	56.3	13.9	1.96	21.3	2.25	11.6	1.71	1880
45	7.00	29.6	348	57.7	5.15	64.2	50.0	12.4	1.95	19.0	2.23	11.5	1.26	1650
40	7.77	33.6	307	51.5	5.13	57.0	44.1	11.0	1.94	16.8	2.21	11.4	0.906	1440
35	6.31	36.2	285	45.6	5.25	51.2	24.5	7.47	1.54	11.5	1.79	12.0	0.741	879
30	7.41	41.8	238	38.6	5.21	43.1	20.3	6.24	1.52	9.56	1.77	11.9	0.457	720
26	8.54	47.2	204	33.4	5.17	37.2	17.3	5.34	1.51	8.17	1.75	11.8	0.300	607
22	4.74	41.8	156	25.4	4.91	29.3	4.66	2.31	0.848	3.66	1.04	11.9	0.293	164
19	5.72	46.2	130	21.3	4.82	24.7	3.76	1.88	0.822	2.98	1.02	11.8	0.180	131
16	7.53	49.4	103	17.1	4.67	20.1	2.82	1.41	0.773	2.26	0.982	11.7	0.103	96.9
14	8.82	54.3	88.6	14.9	4.62	17.4	2.36	1.19	0.753	1.90	0.962	11.7	0.0704	80.4
112	4.17	10.4	716	126	4.66	147	236	45.3	2.68	69.2	3.07	10.1	15.1	6020
100	4.62	11.6	623	112	4.60	130	207	40.0	2.65	61.0	3.03	10.0	10.9	5150
88	5.18	13.0	534	98.5	4.54	113	179	34.8	2.63	53.1	2.99	9.85	7.53	4330
77	5.86	14.8	455	85.9	4.49	97.6	154	30.1	2.60	45.9	2.95	9.73	5.11	3630
68	6.58	16.7	394	75.7	4.44	85.3	134	26.4	2.59	40.1	2.91	9.63	3.56	3100
60	7.41	18.7	341	66.7	4.39	74.6	116	23.0	2.57	35.0	2.88	9.54	2.48	2640
54	8.15	21.2	303	60.0	4.37	66.6	103	20.6	2.56	31.3	2.86	9.48	1.82	2320
49	8.93	23.1	272	54.6	4.35	60.4	93.4	18.7	2.54	28.3	2.84	9.42	1.39	2070
45	6.47	22.5	248	49.1	4.32	54.9	53.4	13.3	2.01	20.3	2.27	9.48	1.51	1200
39	7.53	25.0	209	42.1	4.27	46.8	45.0	11.3	1.98	17.2	2.24	9.39	0.976	992
33	9.15	27.1	171	35.0	4.19	38.8	36.6	9.20	1.94	14.0	2.20	9.30	0.583	791
30	5.70	29.5	170	32.4	4.38	36.6	16.7	5.75	1.37	8.84	1.60	10.0	0.622	414
26	6.56	34.0	144	27.9	4.35	31.3	14.1	4.89	1.36	7.50	1.58	9.89	0.402	345
22	7.99	36.9	118	23.2	4.27	26.0	11.4	3.97	1.33	6.10	1.55	9.81	0.239	275
19	5.09	35.4	96.3	18.8	4.14	21.6	4.29	2.14	0.874	3.35	1.06	9.85	0.233	104
17	6.08	36.9	81.9	16.2	4.05	18.7	3.56	1.78	0.845	2.80	1.04	9.78	0.156	85.1
15	7.41	38.5	68.9	13.8	3.95	16.0	2.89	1.45	0.810	2.30	1.01	9.72	0.104	68.3
12	9.43	46.6	53.8	10.9	3.90	12.6	2.18	1.10	0.785	1.74	0.983	9.66	0.0547	50.9

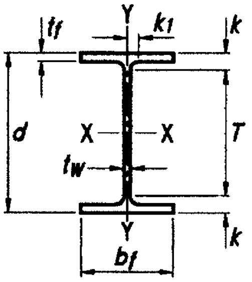


Table 1-1 (continued)
W Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance				
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k		k_1	T	Workable Gage
											k_{des}	k_{det}			
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
W8×67	19.7	9.00	9	0.570	9/16	5/16	8.28	8 1/4	0.935	15/16	1.33	1 5/8	15/16	5 3/4	5 1/2
×58	17.1	8.75	8 3/4	0.510	1/2	1/4	8.22	8 1/4	0.810	13/16	1.20	1 1/2	7/8		
×48	14.1	8.50	8 1/2	0.400	3/8	3/16	8.11	8 1/8	0.685	11/16	1.08	1 3/8	13/16		
×40	11.7	8.25	8 1/4	0.360	3/8	3/16	8.07	8 1/8	0.560	9/16	0.954	1 1/4	13/16		
×35	10.3	8.12	8 1/8	0.310	5/16	3/16	8.02	8	0.495	1/2	0.889	1 3/16	13/16		
×31[f]	9.12	8.00	8	0.285	5/16	3/16	8.00	8	0.435	7/16	0.829	1 1/8	3/4	↓	↓
W8×28	8.24	8.06	8	0.285	5/16	3/16	6.54	6 1/2	0.465	7/16	0.859	15/16	5/8	6 1/8	4
×24	7.08	7.93	7 7/8	0.245	1/4	1/8	6.50	6 1/2	0.400	3/8	0.794	7/8	9/16	6 1/8	4
W8×21	6.16	8.28	8 1/4	0.250	1/4	1/8	5.27	5 1/4	0.400	3/8	0.700	7/8	9/16	6 1/2	2 3/4[g]
×18	5.26	8.14	8 1/8	0.230	1/4	1/8	5.25	5 1/4	0.330	5/16	0.630	13/16	9/16	6 1/2	2 3/4[g]
W8×15	4.44	8.11	8 1/8	0.245	1/4	1/8	4.02	4	0.315	5/16	0.615	13/16	9/16	6 1/2	2 1/4[g]
×13	3.84	7.99	8	0.230	1/4	1/8	4.00	4	0.255	1/4	0.555	3/4	9/16		
×10[c,f]	2.96	7.89	7 7/8	0.170	3/16	1/8	3.94	4	0.205	3/16	0.505	11/16	1/2	↓	↓
W6×25	7.34	6.38	6 3/8	0.320	5/16	3/16	6.08	6 1/8	0.455	7/16	0.705	15/16	9/16	4 1/2	3 1/2
×20	5.87	6.20	6 1/4	0.260	1/4	1/8	6.02	6	0.365	3/8	0.615	7/8	9/16		
×15[f]	4.43	5.99	6	0.230	1/4	1/8	5.99	6	0.260	1/4	0.510	3/4	9/16	↓	↓
W6×16	4.74	6.28	6 1/4	0.260	1/4	1/8	4.03	4	0.405	3/8	0.655	7/8	9/16	4 1/2	2 1/4[g]
×12	3.55	6.03	6	0.230	1/4	1/8	4.00	4	0.280	1/4	0.530	3/4	9/16		
×9[f]	2.68	5.90	5 7/8	0.170	3/16	1/8	3.94	4	0.215	3/16	0.465	11/16	1/2		
×8.5[f]	2.52	5.83	5 7/8	0.170	3/16	1/8	3.94	4	0.195	3/16	0.445	11/16	1/2	↓	↓
W5×19	5.56	5.15	5 1/8	0.270	1/4	1/8	5.03	5	0.430	7/16	0.730	13/16	7/16	3 1/2	2 3/4[g]
×16	4.71	5.01	5	0.240	1/4	1/8	5.00	5	0.360	3/8	0.660	3/4	7/16	3 1/2	2 3/4[g]
W4×13	3.83	4.16	4 1/8	0.280	1/4	1/8	4.06	4	0.345	3/8	0.595	3/4	1/2	2 5/8	2 1/4[g]

[c] Shape is slender for compression with $F_y = 50$ ksi.

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

Table 1–1 (continued)
W Shapes
Properties

W8 – W4

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
67	4.43	11.1	272	60.4	3.72	70.1	88.6	21.4	2.12	32.7	2.43	8.07	5.05	1440
58	5.07	12.4	228	52.0	3.65	59.8	75.1	18.3	2.10	27.9	2.39	7.94	3.33	1180
48	5.92	15.9	184	43.2	3.61	49.0	60.9	15.0	2.08	22.9	2.35	7.82	1.96	931
40	7.21	17.6	146	35.5	3.53	39.8	49.1	12.2	2.04	18.5	2.31	7.69	1.12	726
35	8.10	20.5	127	31.2	3.51	34.7	42.6	10.6	2.03	16.1	2.28	7.63	0.769	619
31	9.19	22.3	110	27.5	3.47	30.4	37.1	9.27	2.02	14.1	2.26	7.57	0.536	530
28	7.03	22.3	98.0	24.3	3.45	27.2	21.7	6.63	1.62	10.1	1.84	7.60	0.537	312
24	8.12	25.9	82.7	20.9	3.42	23.1	18.3	5.63	1.61	8.57	1.82	7.53	0.346	259
21	6.59	27.5	75.3	18.2	3.49	20.4	9.77	3.71	1.26	5.69	1.46	7.88	0.282	152
18	7.95	29.9	61.9	15.2	3.43	17.0	7.97	3.04	1.23	4.66	1.43	7.81	0.172	122
15	6.37	28.1	48.0	11.8	3.29	13.6	3.41	1.70	0.876	2.67	1.06	7.80	0.137	51.8
13	7.84	29.9	39.6	9.91	3.21	11.4	2.73	1.37	0.843	2.15	1.03	7.74	0.0871	40.8
10	9.61	40.5	30.8	7.81	3.22	8.87	2.09	1.06	0.841	1.66	1.01	7.69	0.0426	30.9
25	6.68	15.5	53.4	16.7	2.70	18.9	17.1	5.61	1.52	8.56	1.74	5.93	0.461	150
20	8.25	19.1	41.4	13.4	2.66	14.9	13.3	4.41	1.50	6.72	1.70	5.84	0.240	113
15	11.5	21.6	29.1	9.72	2.56	10.8	9.32	3.11	1.45	4.75	1.66	5.73	0.101	76.5
16	4.98	19.1	32.1	10.2	2.60	11.7	4.43	2.20	0.967	3.39	1.13	5.88	0.223	38.2
12	7.14	21.6	22.1	7.31	2.49	8.30	2.99	1.50	0.918	2.32	1.08	5.75	0.0903	24.7
9	9.16	29.2	16.4	5.56	2.47	6.23	2.20	1.11	0.905	1.72	1.06	5.69	0.0405	17.7
8.5	10.1	29.1	14.9	5.10	2.43	5.73	1.99	1.01	0.890	1.56	1.05	5.64	0.0333	15.8
19	5.85	13.7	26.3	10.2	2.17	11.6	9.13	3.63	1.28	5.53	1.45	4.72	0.316	50.9
16	6.94	15.4	21.4	8.55	2.13	9.63	7.51	3.00	1.26	4.58	1.43	4.65	0.192	40.6
13	5.88	10.6	11.3	5.46	1.72	6.28	3.86	1.90	1.00	2.92	1.16	3.82	0.151	14.0

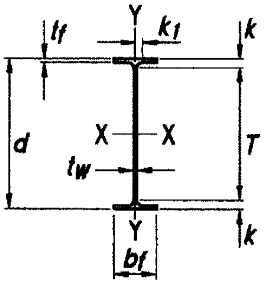


Table 1-2
M Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance			
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.
M12.5×12.4[c,v]	3.63	12.5	12 1/2	0.155	1/8	1/16	3.75	3 3/4	0.228	1/4	9/16	3/8	11 3/8	—
×11.6[c,v]	3.40	12.5	12 1/2	0.155	1/8	1/16	3.50	3 1/2	0.211	3/16	9/16	3/8	11 3/8	—
M12×11.8[c]	3.47	12.0	12	0.177	3/16	1/8	3.07	3 1/8	0.225	1/4	9/16	3/8	10 7/8	—
×10.8[c]	3.18	12.0	12	0.160	3/16	1/8	3.07	3 1/8	0.210	3/16	9/16	3/8	10 7/8	—
M12×10[c,v]	2.95	12.0	12	0.149	1/8	1/16	3.25	3 1/4	0.180	3/16	1/2	3/8	11	—
M10×9[c]	2.65	10.0	10	0.157	3/16	1/8	2.69	2 3/4	0.206	3/16	9/16	3/8	8 7/8	—
×8[c]	2.37	9.95	10	0.141	1/8	1/16	2.69	2 3/4	0.182	3/16	9/16	3/8	8 7/8	—
M10×7.5[c,v]	2.22	9.99	10	0.130	1/8	1/16	2.69	2 3/4	0.173	3/16	7/16	5/16	9 1/8	—
M8×6.5[c]	1.92	8.00	8	0.135	1/8	1/16	2.28	2 1/4	0.189	3/16	9/16	3/8	6 7/8	—
×6.2[c]	1.82	8.00	8	0.129	1/8	1/16	2.28	2 1/4	0.177	3/16	7/16	1/4	7 1/8	—
M6×4.4[c]	1.29	6.00	6	0.114	1/8	1/16	1.84	1 7/8	0.171	3/16	3/8	1/4	5 1/4	—
×3.7[c]	1.09	5.92	5 7/8	0.0980	1/8	1/16	2.00	2	0.129	1/8	5/16	1/4	5 1/4	—
M5×18.9[t]	5.56	5.00	5	0.316	5/16	3/16	5.00	5	0.416	7/16	13/16	1/2	3 3/8	2 3/4[g]
M4×6[f]	1.75	3.80	3 3/4	0.130	1/8	1/16	3.80	3 3/4	0.160	3/16	1/2	3/8	2 3/4	—
×4.08	1.27	4.00	4	0.115	1/8	1/16	2.25	2 1/4	0.170	3/16	9/16	3/8	2 7/8	—
×3.45	1.01	4.00	4	0.0920	1/16	1/16	2.25	2 1/4	0.130	1/8	1/2	3/8	3	—
×3.2	1.01	4.00	4	0.0920	1/16	1/16	2.25	2 1/4	0.130	1/8	1/2	3/8	3	—
M3×2.9	0.914	3.00	3	0.0900	1/16	1/16	2.25	2 1/4	0.130	1/8	1/2	3/8	2	—

[c] Shape is slender for compression with $F_y = 36$ ksi.

[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[t] Shape has tapered flanges while other M-shapes have parallel flange surfaces.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1b(i) with $F_y = 36$ ksi.

— Flange is too narrow to establish a workable gage.

Table 1–2 (continued)
M Shapes
Properties

M SHAPES

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	$\frac{J}{S_x h_o}$	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z				J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.		in.^{4}	in.^{6}
12.4	8.22	74.8	89.3	14.2	4.96	16.5	2.01	1.07	0.744	1.68	0.100	12.3	0.000283	0.0493	76.0
11.6	8.29	74.8	80.3	12.8	4.86	15.0	1.51	0.864	0.667	1.37	0.099	12.3	0.000263	0.0414	57.1
11.8	6.81	62.5	72.2	12.0	4.56	14.3	1.09	0.709	0.559	1.15	0.108	11.8	0.000355	0.0500	37.7
10.8	7.30	69.2	66.7	11.1	4.58	13.2	1.01	0.661	0.564	1.07	0.104	11.8	0.000300	0.0393	35.0
10	9.03	74.7	61.7	10.3	4.57	12.2	1.03	0.636	0.592	1.02	0.098	11.8	0.000240	0.0292	35.9
9	6.53	58.4	39.0	7.79	3.83	9.22	0.672	0.500	0.503	0.809	0.117	9.81	0.000411	0.0314	16.1
8	7.39	65.0	34.6	6.95	3.82	8.20	0.593	0.441	0.500	0.711	0.111	9.81	0.000328	0.0224	14.2
7.5	7.77	71.0	33.0	6.60	3.85	7.77	0.562	0.418	0.503	0.670	0.107	9.81	0.000289	0.0187	13.5
6.5	6.03	53.8	18.5	4.63	3.11	5.43	0.376	0.329	0.443	0.529	0.131	7.81	0.000509	0.0184	5.73
6.2	6.44	56.5	17.6	4.39	3.10	5.15	0.352	0.308	0.439	0.495	0.127	7.81	0.000455	0.0156	5.38
4.4	5.39	47.0	7.23	2.41	2.36	2.80	0.180	0.195	0.372	0.311	0.152	5.81	0.000707	0.00990	1.53
3.7	7.75	54.7	5.96	2.01	2.34	2.33	0.173	0.173	0.398	0.273	0.137	5.75	0.000459	0.00530	1.45
18.9	6.01	11.2	24.2	9.67	2.08	11.1	8.70	3.48	1.25	5.33	0.28	4.56	0.00709	0.313	45.7
6	11.9	22.0	4.72	2.48	1.64	2.74	1.47	0.771	0.915	1.18	0.22	3.56	0.00208	0.0184	4.85
4.08	6.62	26.4	3.53	1.77	1.67	2.00	0.325	0.289	0.506	0.453	0.220	3.81	0.00218	0.0147	1.19
3.45	8.65	33.9	2.86	1.43	1.68	1.60	0.248	0.221	0.496	0.346	0.200	3.88	0.00148	0.00820	0.930
3.2	8.65	33.9	2.86	1.43	1.68	1.60	0.248	0.221	0.496	0.346	0.200	3.88	0.00148	0.00820	0.930
2.9	8.65	23.6	1.50	1.00	1.28	1.12	0.248	0.221	0.521	0.344	0.250	2.88	0.00275	0.00790	0.511

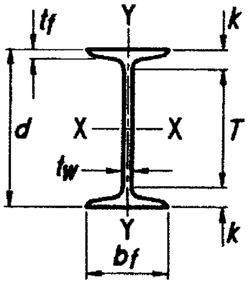


Table 1–3
S Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.
S24×121	35.5	24.5	$24\frac{1}{2}$	0.800	$\frac{13}{16}$	$\frac{7}{16}$	8.05	8	1.09	$1\frac{1}{16}$	2	$20\frac{1}{2}$	4
×106	31.1	24.5	$24\frac{1}{2}$	0.620	$\frac{5}{8}$	$\frac{5}{16}$	7.87	$7\frac{7}{8}$	1.09	$1\frac{1}{16}$	2	$20\frac{1}{2}$	4
S24×100	29.3	24.0	24	0.745	$\frac{3}{4}$	$\frac{3}{8}$	7.25	$7\frac{1}{4}$	0.870	$\frac{7}{8}$	$1\frac{3}{4}$	$20\frac{1}{2}$	4
×90	26.5	24.0	24	0.625	$\frac{5}{8}$	$\frac{5}{16}$	7.13	$7\frac{1}{8}$	0.870	$\frac{7}{8}$	$1\frac{3}{4}$	$20\frac{1}{2}$	4
×80	23.5	24.0	24	0.500	$\frac{1}{2}$	$\frac{1}{4}$	7.00	7	0.870	$\frac{7}{8}$	$1\frac{3}{4}$	$20\frac{1}{2}$	4
S20×96	28.2	20.3	$20\frac{1}{4}$	0.800	$\frac{13}{16}$	$\frac{7}{16}$	7.20	$7\frac{1}{4}$	0.920	$\frac{15}{16}$	$1\frac{3}{4}$	$16\frac{3}{4}$	4
×86	25.3	20.3	$20\frac{1}{4}$	0.660	$\frac{11}{16}$	$\frac{3}{8}$	7.06	7	0.920	$\frac{15}{16}$	$1\frac{3}{4}$	$16\frac{3}{4}$	4
S20×75	22.0	20.0	20	0.635	$\frac{5}{8}$	$\frac{5}{16}$	6.39	$6\frac{3}{8}$	0.795	$\frac{13}{16}$	$1\frac{5}{8}$	$16\frac{3}{4}$	$3\frac{1}{2}$[g]
×66	19.4	20.0	20	0.505	$\frac{1}{2}$	$\frac{1}{4}$	6.26	$6\frac{1}{4}$	0.795	$\frac{13}{16}$	$1\frac{5}{8}$	$16\frac{3}{4}$	$3\frac{1}{2}$[g]
S18×70	20.5	18.0	18	0.711	$\frac{11}{16}$	$\frac{3}{8}$	6.25	$6\frac{1}{4}$	0.691	$\frac{11}{16}$	$1\frac{1}{2}$	15	$3\frac{1}{2}$[g]
×54.7	16.0	18.0	18	0.461	$\frac{7}{16}$	$\frac{1}{4}$	6.00	6	0.691	$\frac{11}{16}$	$1\frac{1}{2}$	15	$3\frac{1}{2}$[g]
S15×50	14.7	15.0	15	0.550	$\frac{9}{16}$	$\frac{5}{16}$	5.64	$5\frac{5}{8}$	0.622	$\frac{5}{8}$	$1\frac{3}{8}$	$12\frac{1}{4}$	$3\frac{1}{2}$[g]
×42.9	12.6	15.0	15	0.411	$\frac{7}{16}$	$\frac{1}{4}$	5.50	$5\frac{1}{2}$	0.622	$\frac{5}{8}$	$1\frac{3}{8}$	$12\frac{1}{4}$	$3\frac{1}{2}$[g]
S12×50	14.6	12.0	12	0.687	$\frac{11}{16}$	$\frac{3}{8}$	5.48	$5\frac{1}{2}$	0.659	$\frac{11}{16}$	$1\frac{7}{16}$	$9\frac{1}{8}$	3[g]
×40.8	11.9	12.0	12	0.462	$\frac{7}{16}$	$\frac{1}{4}$	5.25	$5\frac{1}{4}$	0.659	$\frac{11}{16}$	$1\frac{7}{16}$	$9\frac{1}{8}$	3[g]
S12×35	10.2	12.0	12	0.428	$\frac{7}{16}$	$\frac{1}{4}$	5.08	$5\frac{1}{8}$	0.544	$\frac{9}{16}$	$1\frac{3}{16}$	$9\frac{5}{8}$	3[g]
×31.8	9.31	12.0	12	0.350	$\frac{3}{8}$	$\frac{3}{16}$	5.00	5	0.544	$\frac{9}{16}$	$1\frac{3}{16}$	$9\frac{5}{8}$	3[g]
S10×35	10.3	10.0	10	0.594	$\frac{5}{8}$	$\frac{5}{16}$	4.94	5	0.491	$\frac{1}{2}$	$1\frac{1}{8}$	$7\frac{3}{4}$	$2\frac{3}{4}$[g]
×25.4	7.45	10.0	10	0.311	$\frac{5}{16}$	$\frac{3}{16}$	4.66	$4\frac{5}{8}$	0.491	$\frac{1}{2}$	$1\frac{1}{8}$	$7\frac{3}{4}$	$2\frac{3}{4}$[g]
S8×23	6.76	8.00	8	0.441	$\frac{7}{16}$	$\frac{1}{4}$	4.17	$4\frac{1}{8}$	0.425	$\frac{7}{16}$	1	6	$2\frac{1}{4}$[g]
×18.4	5.40	8.00	8	0.271	$\frac{1}{4}$	$\frac{1}{8}$	4.00	4	0.425	$\frac{7}{16}$	1	6	$2\frac{1}{4}$[g]
S6×17.2	5.06	6.00	6	0.465	$\frac{7}{16}$	$\frac{1}{4}$	3.57	$3\frac{5}{8}$	0.359	$\frac{3}{8}$	$\frac{13}{16}$	$4\frac{3}{8}$	—
×12.5	3.66	6.00	6	0.232	$\frac{1}{4}$	$\frac{1}{8}$	3.33	$3\frac{3}{8}$	0.359	$\frac{3}{8}$	$\frac{13}{16}$	$4\frac{3}{8}$	—
S5×10	2.93	5.00	5	0.214	$\frac{3}{16}$	$\frac{1}{8}$	3.00	3	0.326	$\frac{5}{16}$	$\frac{3}{4}$	$3\frac{1}{2}$	—
S4×9.5	2.79	4.00	4	0.326	$\frac{5}{16}$	$\frac{3}{16}$	2.80	$2\frac{3}{4}$	0.293	$\frac{5}{16}$	$\frac{3}{4}$	$2\frac{1}{2}$	—
×7.7	2.26	4.00	4	0.193	$\frac{3}{16}$	$\frac{1}{8}$	2.66	$2\frac{5}{8}$	0.293	$\frac{5}{16}$	$\frac{3}{4}$	$2\frac{1}{2}$	—
S3×7.5	2.20	3.00	3	0.349	$\frac{3}{8}$	$\frac{3}{16}$	2.51	$2\frac{1}{2}$	0.260	$\frac{1}{4}$	$\frac{5}{8}$	$1\frac{3}{4}$	—
×5.7	1.66	3.00	3	0.170	$\frac{3}{16}$	$\frac{1}{8}$	2.33	$2\frac{3}{8}$	0.260	$\frac{1}{4}$	$\frac{5}{8}$	$1\frac{3}{4}$	—

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

— Flange is too narrow to establish a workable gage.

Table 1–3 (continued)
S Shapes
Properties

S SHAPES

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z			J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{6}
121	3.69	25.9	3160	258	9.43	306	83.0	20.6	1.53	36.3	1.94	23.4	12.8	11400
106	3.61	33.4	2940	240	9.71	279	76.8	19.5	1.57	33.4	1.93	23.4	10.1	10500
100	4.16	27.8	2380	199	9.01	239	47.4	13.1	1.27	24.0	1.66	23.1	7.59	6350
90	4.09	33.1	2250	187	9.21	222	44.7	12.5	1.30	22.4	1.66	23.1	6.05	5980
80	4.02	41.4	2100	175	9.47	204	42.0	12.0	1.34	20.8	1.67	23.1	4.89	5620
96	3.91	21.1	1670	165	7.71	198	49.9	13.9	1.33	24.9	1.71	19.4	8.40	4690
86	3.84	25.6	1570	155	7.89	183	46.6	13.2	1.36	23.1	1.71	19.4	6.65	4370
75	4.02	26.6	1280	128	7.62	152	29.5	9.25	1.16	16.7	1.49	19.2	4.59	2720
66	3.93	33.5	1190	119	7.83	139	27.5	8.78	1.19	15.4	1.49	19.2	3.58	2530
70	4.52	21.5	923	103	6.70	124	24.0	7.69	1.08	14.3	1.42	17.3	4.10	1800
54.7	4.34	33.2	801	89.0	7.07	104	20.7	6.91	1.14	12.1	1.42	17.3	2.33	1550
50	4.53	22.7	485	64.7	5.75	77.0	15.6	5.53	1.03	9.99	1.32	14.4	2.12	805
42.9	4.42	30.4	446	59.4	5.95	69.2	14.3	5.19	1.06	9.08	1.31	14.4	1.54	737
50	4.16	13.7	303	50.6	4.55	60.9	15.6	5.69	1.03	10.3	1.32	11.3	2.77	501
40.8	3.98	20.6	270	45.1	4.76	52.7	13.5	5.13	1.06	8.86	1.30	11.3	1.69	433
35	4.67	23.1	228	38.1	4.72	44.6	9.84	3.88	0.980	6.80	1.22	11.5	1.05	323
31.8	4.60	28.3	217	36.2	4.83	41.8	9.33	3.73	1.00	6.44	1.21	11.5	0.878	306
35	5.03	13.4	147	29.4	3.78	35.4	8.30	3.36	0.899	6.19	1.16	9.51	1.29	188
25.4	4.75	25.6	123	24.6	4.07	28.3	6.73	2.89	0.950	4.99	1.14	9.51	0.603	152
23	4.91	14.1	64.7	16.2	3.09	19.2	4.27	2.05	0.795	3.67	0.999	7.58	0.550	61.2
18.4	4.71	22.9	57.5	14.4	3.26	16.5	3.69	1.84	0.827	3.18	0.985	7.58	0.335	52.9
17.2	4.97	9.67	26.2	8.74	2.28	10.5	2.29	1.28	0.673	2.35	0.859	5.64	0.371	18.2
12.5	4.64	19.4	22.0	7.34	2.45	8.45	1.80	1.08	0.702	1.86	0.831	5.64	0.167	14.3
10	4.61	16.8	12.3	4.90	2.05	5.66	1.19	0.795	0.638	1.37	0.754	4.67	0.114	6.52
9.5	4.77	8.33	6.76	3.38	1.56	4.04	0.887	0.635	0.564	1.13	0.698	3.71	0.120	3.05
7.7	4.54	14.1	6.05	3.03	1.64	3.50	0.748	0.562	0.576	0.970	0.676	3.71	0.0732	2.57
7.5	4.83	5.38	2.91	1.94	1.15	2.35	0.578	0.461	0.513	0.821	0.638	2.74	0.0896	1.08
5.7	4.48	11.0	2.50	1.67	1.23	1.94	0.447	0.383	0.518	0.656	0.605	2.74	0.0433	0.838

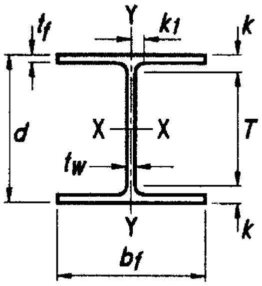


Table 1–4

HP Shapes

Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance			
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k	k_1	T	Workable Gage
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.
HP14×117^{f}	34.4	14.2	$14\frac{1}{4}$	0.805	$\frac{13}{16}$	$\frac{7}{16}$	14.9	$14\frac{7}{8}$	0.805	$\frac{13}{16}$	$1\frac{1}{2}$	$1\frac{1}{16}$	$11\frac{1}{4}$	$5\frac{1}{2}$
×102^{f}	30.0	14.0	14	0.705	$\frac{11}{16}$	$\frac{3}{8}$	14.8	$14\frac{3}{4}$	0.705	$\frac{11}{16}$	$1\frac{3}{8}$	1	↓	↓
×89^{f}	26.1	13.8	$13\frac{7}{8}$	0.615	$\frac{5}{8}$	$\frac{5}{16}$	14.7	$14\frac{3}{4}$	0.615	$\frac{5}{8}$	$1\frac{5}{16}$	$\frac{15}{16}$	↓	↓
×73^{c,f}	21.4	13.6	$13\frac{5}{8}$	0.505	$\frac{1}{2}$	$\frac{1}{4}$	14.6	$14\frac{5}{8}$	0.505	$\frac{1}{2}$	$1\frac{3}{16}$	$\frac{7}{8}$	↓	↓
HP12×84	24.6	12.3	$12\frac{1}{4}$	0.685	$\frac{11}{16}$	$\frac{3}{8}$	12.3	$12\frac{1}{4}$	0.685	$\frac{11}{16}$	$1\frac{3}{8}$	1	$9\frac{1}{2}$	$5\frac{1}{2}$
×74^{f}	21.8	12.1	$12\frac{1}{8}$	0.605	$\frac{5}{8}$	$\frac{5}{16}$	12.2	$12\frac{1}{4}$	0.610	$\frac{5}{8}$	$1\frac{5}{16}$	$\frac{15}{16}$	↓	↓
×63^{f}	18.4	11.9	12	0.515	$\frac{1}{2}$	$\frac{1}{4}$	12.1	$12\frac{1}{8}$	0.515	$\frac{1}{2}$	$1\frac{1}{4}$	$\frac{7}{8}$	↓	↓
×53^{f}	15.5	11.8	$11\frac{3}{4}$	0.435	$\frac{7}{16}$	$\frac{1}{4}$	12.0	12	0.435	$\frac{7}{16}$	$1\frac{1}{8}$	$\frac{7}{8}$	↓	↓
HP10×57^{f}	16.8	9.99	10	0.565	$\frac{9}{16}$	$\frac{5}{16}$	10.2	$10\frac{1}{4}$	0.565	$\frac{9}{16}$	$1\frac{1}{4}$	$\frac{15}{16}$	$7\frac{1}{2}$	$5\frac{1}{2}$
×42^{f}	12.4	9.70	$9\frac{3}{4}$	0.415	$\frac{7}{16}$	$\frac{1}{4}$	10.1	$10\frac{1}{8}$	0.420	$\frac{7}{16}$	$1\frac{1}{8}$	$\frac{13}{16}$	$7\frac{1}{2}$	$5\frac{1}{2}$
HP8×36^{f}	10.6	8.02	8	0.445	$\frac{7}{16}$	$\frac{1}{4}$	8.16	$8\frac{1}{8}$	0.445	$\frac{7}{16}$	$1\frac{1}{8}$	$\frac{7}{8}$	$5\frac{3}{4}$	$5\frac{1}{2}$

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

Table 1–4 (continued)
HP Shapes
Properties

HP SHAPES

Nom-inal Wt.	Compact Section Criteria		Axis X-X				Axis Y-Y				r_{ts}	h_o	$\frac{J}{S_x h_o}$	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	Z	I	S	r	Z				J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.		in.^{4}	in.^{6}
117	9.25	14.2	1220	172	5.96	194	443	59.5	3.59	91.4	4.15	13.41	0.00348	8.02	19900
102	10.5	16.2	1050	150	5.92	169	380	51.4	3.56	78.8	4.10	13.31	0.00270	5.39	16800
89	11.9	18.5	904	131	5.88	146	326	44.3	3.53	67.7	4.05	13.22	0.00207	3.59	14200
73	14.4	22.6	729	107	5.84	118	261	35.8	3.49	54.6	4.00	13.11	0.00143	2.01	11200
84	8.97	14.2	650	106	5.14	120	213	34.6	2.94	53.2	3.41	11.60	0.00345	4.24	7140
74	10.0	16.1	569	93.8	5.11	105	186	30.4	2.92	46.6	3.38	11.52	0.00276	2.98	6160
63	11.8	18.9	472	79.1	5.06	88.3	153	25.3	2.88	38.7	3.33	11.43	0.00202	1.83	5000
53	13.8	22.3	393	66.7	5.03	74.0	127	21.1	2.86	32.2	3.29	11.35	0.00148	1.12	4080
57	9.05	13.9	294	58.8	4.18	66.5	101	19.7	2.45	30.3	2.84	9.43	0.00355	1.97	2240
42	12.0	18.9	210	43.4	4.13	48.3	71.7	14.2	2.41	21.8	2.77	9.28	0.00202	0.813	1540
36	9.16	14.2	119	29.8	3.36	33.6	40.3	9.88	1.95	15.2	2.26	7.58	0.00341	0.770	578

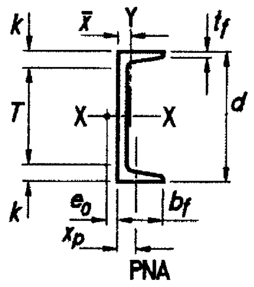


Table 1-5
C Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance			r_{ts}	h_o
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Thickness, t_f		k	T	Workable Gage		
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
C15×50	14.7	15.0	15	0.716	11/16	3/8	3.72	3 3/4	0.650	5/8	1 7/16	12 1/8	2 1/4	1.17	14.4
×40	11.8	15.0	15	0.520	1/2	1/4	3.52	3 1/2	0.650	5/8	1 7/16	12 1/8	2	1.15	14.4
×33.9	10.0	15.0	15	0.400	3/8	3/16	3.40	3 3/8	0.650	5/8	1 7/16	12 1/8	2	1.13	14.4
C12×30	8.81	12.0	12	0.510	1/2	1/4	3.17	3 1/8	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	1.01	11.5
×25	7.34	12.0	12	0.387	3/8	3/16	3.05	3	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	1.00	11.5
×20.7	6.08	12.0	12	0.282	5/16	3/16	2.94	3	0.501	1/2	1 1/8	9 3/4	1 3/4[g]	0.983	11.5
C10×30	8.81	10.0	10	0.673	11/16	3/8	3.03	3	0.436	7/16	1	8	1 3/4[g]	0.925	9.56
×25	7.34	10.0	10	0.526	1/2	1/4	2.89	2 7/8	0.436	7/16	1	8	1 3/4[g]	0.911	9.56
×20	5.87	10.0	10	0.379	3/8	3/16	2.74	2 3/4	0.436	7/16	1	8	1 1/2[g]	0.894	9.56
×15.3	4.48	10.0	10	0.240	1/4	1/8	2.60	2 5/8	0.436	7/16	1	8	1 1/2[g]	0.869	9.56
C9×20	5.87	9.00	9	0.448	7/16	1/4	2.65	2 5/8	0.413	7/16	1	7	1 1/2[g]	0.848	8.59
×15	4.41	9.00	9	0.285	5/16	3/16	2.49	2 1/2	0.413	7/16	1	7	1 3/8[g]	0.824	8.59
×13.4	3.94	9.00	9	0.233	1/4	1/8	2.43	2 3/8	0.413	7/16	1	7	1 3/8[g]	0.813	8.59
C8×18.7	5.51	8.00	8	0.487	1/2	1/4	2.53	2 1/2	0.390	3/8	15/16	6 1/8	1 1/2[g]	0.800	7.61
×13.7	4.04	8.00	8	0.303	5/16	3/16	2.34	2 3/8	0.390	3/8	15/16	6 1/8	1 3/8[g]	0.774	7.61
×11.5	3.37	8.00	8	0.220	1/4	1/8	2.26	2 1/4	0.390	3/8	15/16	6 1/8	1 3/8[g]	0.756	7.61
C7×14.7	4.33	7.00	7	0.419	7/16	1/4	2.30	2 1/4	0.366	3/8	7/8	5 1/4	1 1/4[g]	0.738	6.63
×12.2	3.60	7.00	7	0.314	5/16	3/16	2.19	2 1/4	0.366	3/8	7/8	5 1/4	1 1/4[g]	0.721	6.63
×9.8	2.87	7.00	7	0.210	3/16	1/8	2.09	2 1/8	0.366	3/8	7/8	5 1/4	1 1/4[g]	0.698	6.63
C6×13	3.81	6.00	6	0.437	7/16	1/4	2.16	2 1/8	0.343	5/16	13/16	4 /8	1 3/8[g]	0.689	5.66
×10.5	3.08	6.00	6	0.314	5/16	3/16	2.03	2	0.343	5/16	13/16	4 3/8	1 1/8[g]	0.669	5.66
×8.2	2.39	6.00	6	0.200	3/16	1/8	1.92	1 7/8	0.343	5/16	13/16	4 3/8	1 1/8[g]	0.643	5.66
C5×9	2.64	5.00	5	0.325	5/16	3/16	1.89	1 7/8	0.320	5/16	3/4	3 1/2	1 1/8[g]	0.617	4.68
×6.7	1.97	5.00	5	0.190	3/16	1/8	1.75	1 3/4	0.320	5/16	3/4	3 1/2	—	0.584	4.68
C4×7.2	2.13	4.00	4	0.321	5/16	3/16	1.72	1 3/4	0.296	5/16	3/4	2 1/2	1[g]	0.563	3.70
×5.4	1.58	4.00	4	0.184	3/16	1/8	1.58	1 5/8	0.296	5/16	3/4	2 1/2	—	0.528	3.70
×4.5	1.38	4.00	4	0.125	1/8	1/16	1.58	1 5/8	0.296	5/16	3/4	2 1/2	—	0.524	3.70
C3×6	1.76	3.00	3	0.356	3/8	3/16	1.60	1 5/8	0.273	1/4	11/16	1 5/8	—	0.519	2.73
×5	1.47	3.00	3	0.258	1/4	1/8	1.50	1 1/2	0.273	1/4	11/16	1 5/8	—	0.495	2.73
×4.1	1.20	3.00	3	0.170	3/16	1/8	1.41	1 3/8	0.273	1/4	11/16	1 5/8	—	0.469	2.73
×3.5	1.09	3.00	3	0.132	1/8	1/16	1.37	1 3/8	0.273	1/4	11/16	1 5/8	—	0.455	2.73

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

— Flange is too narrow to establish a workable gage.

Table 1–5 (continued)
C Shapes
Properties

C SHAPES

Nominal Wt.	Shear Ctr, e_o	Axis X-X				Axis Y-Y						Torsional Properties			
		I	S	r	Z	I	S	r	$\bar{x}$	Z	x_p	J	C_w	$\bar{r}_o$	H
lb/ft	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.	
50	0.583	404	53.8	5.24	68.5	11.0	3.77	0.865	0.799	8.14	0.490	2.65	492	5.49	0.937
40	0.767	348	46.5	5.45	57.5	9.17	3.34	0.883	0.778	6.84	0.392	1.45	410	5.73	0.927
33.9	0.896	315	42.0	5.62	50.8	8.07	3.09	0.901	0.788	6.19	0.332	1.01	358	5.94	0.920
30	0.618	162	27.0	4.29	33.8	5.12	2.05	0.762	0.674	4.32	0.367	0.861	151	4.54	0.919
25	0.746	144	24.0	4.43	29.4	4.45	1.87	0.779	0.674	3.82	0.306	0.538	130	4.72	0.909
20.7	0.870	129	21.5	4.61	25.6	3.86	1.72	0.797	0.698	3.47	0.253	0.369	112	4.93	0.899
30	0.368	103	20.7	3.42	26.7	3.93	1.65	0.668	0.649	3.78	0.441	1.22	79.5	3.63	0.922
25	0.494	91.1	18.2	3.52	23.1	3.34	1.47	0.675	0.617	3.18	0.367	0.687	68.3	3.75	0.912
20	0.636	78.9	15.8	3.66	19.4	2.80	1.31	0.690	0.606	2.70	0.294	0.368	56.9	3.93	0.900
15.3	0.796	67.3	13.5	3.87	15.9	2.27	1.15	0.711	0.634	2.34	0.224	0.209	45.5	4.19	0.884
20	0.515	60.9	13.5	3.22	16.9	2.41	1.17	0.640	0.583	2.46	0.326	0.427	39.4	3.46	0.899
15	0.681	51.0	11.3	3.40	13.6	1.91	1.01	0.659	0.586	2.04	0.245	0.208	31.0	3.69	0.882
13.4	0.742	47.8	10.6	3.49	12.6	1.75	0.954	0.666	0.601	1.94	0.219	0.168	28.2	3.79	0.875
18.7	0.431	43.9	11.0	2.82	13.9	1.97	1.01	0.598	0.565	2.17	0.344	0.434	25.1	3.05	0.894
13.7	0.604	36.1	9.02	2.99	11.0	1.52	0.848	0.613	0.554	1.73	0.252	0.186	19.2	3.26	0.874
11.5	0.697	32.5	8.14	3.11	9.63	1.31	0.775	0.623	0.572	1.57	0.211	0.130	16.5	3.41	0.862
14.7	0.441	27.2	7.78	2.51	9.75	1.37	0.772	0.561	0.532	1.63	0.309	0.267	13.1	2.75	0.875
12.2	0.538	24.2	6.92	2.60	8.46	1.16	0.696	0.568	0.525	1.42	0.257	0.161	11.2	2.86	0.862
9.8	0.647	21.2	6.07	2.72	7.19	0.957	0.617	0.578	0.541	1.26	0.205	0.0996	9.15	3.03	0.846
13	0.380	17.3	5.78	2.13	7.29	1.05	0.638	0.524	0.514	1.35	0.318	0.237	7.19	2.37	0.858
10.5	0.486	15.1	5.04	2.22	6.18	0.860	0.561	0.529	0.500	1.14	0.256	0.128	5.91	2.48	0.842
8.2	0.599	13.1	4.35	2.34	5.16	0.687	0.488	0.536	0.512	0.987	0.199	0.0736	4.70	2.64	0.823
9	0.427	8.89	3.56	1.83	4.39	0.624	0.444	0.486	0.478	0.913	0.264	0.109	2.93	2.10	0.815
6.7	0.552	7.48	2.99	1.95	3.55	0.470	0.372	0.489	0.484	0.757	0.215	0.0549	2.22	2.26	0.791
7.2	0.386	4.58	2.29	1.47	2.84	0.425	0.337	0.447	0.459	0.695	0.266	0.0817	1.24	1.75	0.767
5.4	0.501	3.85	1.92	1.56	2.29	0.312	0.277	0.444	0.457	0.565	0.231	0.0399	0.921	1.88	0.741
4.5	0.587	3.65	1.83	1.63	2.12	0.289	0.265	0.457	0.493	0.531	0.321	0.0322	0.871	2.00	0.710
6	0.322	2.07	1.38	1.08	1.74	0.300	0.263	0.413	0.455	0.543	0.294	0.0725	0.462	1.40	0.690
5	0.392	1.85	1.23	1.12	1.52	0.241	0.228	0.405	0.439	0.464	0.245	0.0425	0.379	1.45	0.674
4.1	0.461	1.65	1.10	1.17	1.32	0.191	0.196	0.398	0.437	0.399	0.262	0.0269	0.307	1.53	0.655
3.5	0.493	1.57	1.04	1.20	1.24	0.169	0.182	0.394	0.443	0.364	0.296	0.0226	0.276	1.57	0.645

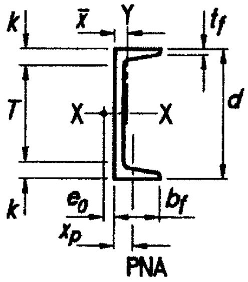


Table 1-6
MC Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance			r_{ts}	h_o
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Average Thickness, t_f		k	T	Workable Gage		
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
MC18×58	17.1	18.0	18	0.700	11/16	3/8	4.20	4 1/4	0.625	5/8	1 7/16	15 1/8	2 1/2	1.35	17.4
×51.9	15.3	18.0	18	0.600	5/8	5/16	4.10	4 1/8	0.625	5/8	1 7/16	↓	↓	1.35	17.4
×45.8	13.5	18.0	18	0.500	1/2	1/4	4.00	4	0.625	5/8	1 7/16	↓	↓	1.34	17.4
×42.7	12.6	18.0	18	0.450	7/16	1/4	3.95	4	0.625	5/8	1 7/16	↓	↓	1.34	17.4
MC13×50	14.7	13.0	13	0.787	13/16	7/16	4.41	4 3/8	0.610	5/8	1 7/16	10 1/8	2 1/2	1.41	12.4
×40	11.8	13.0	13	0.560	9/16	5/16	4.19	4 1/8	0.610	5/8	1 7/16	↓	↓	1.38	12.4
×35	10.3	13.0	13	0.447	7/16	1/4	4.07	4 1/8	0.610	5/8	1 7/16	↓	↓	1.35	12.4
×31.8	9.35	13.0	13	0.375	3/8	3/16	4.00	4	0.610	5/8	1 7/16	↓	↓	1.34	12.4
MC12×50	14.7	12.0	12	0.835	13/16	7/16	4.14	4 1/8	0.700	11/16	1 5/16	9 3/8	2 1/2	1.37	11.3
×45	13.2	12.0	12	0.710	11/16	3/8	4.01	4	0.700	11/16	1 5/16	↓	↓	1.35	11.3
×40	11.8	12.0	12	0.590	9/16	5/16	3.89	3 7/8	0.700	11/16	1 5/16	↓	↓	1.33	11.3
×35	10.3	12.0	12	0.465	7/16	1/4	3.77	3 3/4	0.700	11/16	1 5/16	↓	↓	1.30	11.3
×31	9.12	12.0	12	0.370	3/8	3/16	3.67	3 5/8	0.700	11/16	1 5/16	↓	2 1/4	1.28	11.3
MC12×10.6[c]	3.10	12.0	12	0.190	3/16	1/8	1.50	1 1/2	0.309	5/16	3/4	10 1/2	—	0.477	11.7
MC10×41.1	12.1	10.0	10	0.796	13/16	7/16	4.32	4 3/8	0.575	9/16	1 5/16	7 3/8	2 1/2[g]	1.44	9.43
×33.6	9.87	10.0	10	0.575	9/16	5/16	4.10	4 1/8	0.575	9/16	1 5/16	7 3/8	2 1/2[g]	1.40	9.43
×28.5	8.37	10.0	10	0.425	7/16	1/4	3.95	4	0.575	9/16	1 5/16	7 3/8	2 1/2[g]	1.36	9.43
MC10×25	7.35	10.0	10	0.380	3/8	3/16	3.41	3 3/8	0.575	9/16	1 5/16	7 3/8	2[g]	1.17	9.43
×22	6.45	10.0	10	0.290	5/16	3/16	3.32	3 3/8	0.575	9/16	1 5/16	7 3/8	2[g]	1.14	9.43
MC10×8.4[c]	2.46	10.0	10	0.170	3/16	1/8	1.50	1 1/2	0.280	1/4	3/4	8 1/2	—	0.486	9.72
×6.5[c]	1.95	10.0	10	0.152	1/8	1/16	1.17	1 1/8	0.202	3/16	9/16	8 7/8	—	0.364	9.80
MC9×25.4	7.47	9.00	9	0.450	7/16	1/4	3.50	3 1/2	0.550	9/16	1 1/4	6 1/2	2[g]	1.20	8.45
×23.9	7.02	9.00	9	0.400	3/8	3/16	3.45	3 1/2	0.550	9/16	1 1/4	6 1/2	2[g]	1.18	8.45
MC8×22.8	6.70	8.00	8	0.427	7/16	1/4	3.50	3 1/2	0.525	1/2	1 3/16	5 5/8	2[g]	1.20	7.48
×21.4	6.28	8.00	8	0.375	3/8	3/16	3.45	3 1/2	0.525	1/2	1 3/16	5 5/8	2[g]	1.18	7.48
MC8×20	5.88	8.00	8	0.400	3/8	3/16	3.03	3	0.500	1/2	1 1/8	5 3/4	2[g]	1.03	7.50
×18.7	5.50	8.00	8	0.353	3/8	3/16	2.98	3	0.500	1/2	1 1/8	5 3/4	2[g]	1.02	7.50
MC8×8.5	2.50	8.00	8	0.179	3/16	1/8	1.87	1 7/8	0.311	5/16	13/16	6 3/8	1 1/8[g]	0.624	7.69

[c] Shape is slender for compression with F_y = 36 ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

— Flange is too narrow to establish a workable gage.

Table 1–6 (continued)
MC Shapes
Properties

MC18–MC8

Nom-inal Wt.	Shear Ctr, e_o	Axis X-X				Axis Y-Y						Torsional Properties		
		I	S	r	Z	I	S	r	$\bar{x}$	Z	x_p	J	C_w	$\bar{r}_o$
lb/ft	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
58	0.695	675	75.0	6.29	95.4	17.6	5.28	1.02	0.862	10.7	0.474	2.81	1070	6.56
51.9	0.797	627	69.6	6.41	87.3	16.3	5.02	1.03	0.858	9.86	0.424	2.03	985	6.70
45.8	0.909	578	64.2	6.55	79.2	14.9	4.77	1.05	0.866	9.14	0.374	1.45	897	6.87
42.7	0.969	554	61.5	6.64	75.1	14.3	4.64	1.07	0.877	8.82	0.349	1.23	852	6.97
50	0.815	314	48.3	4.62	60.8	16.4	4.77	1.06	0.974	10.2	0.566	2.96	558	5.07
40	1.03	273	41.9	4.82	51.2	13.7	4.24	1.08	0.963	8.66	0.452	1.55	462	5.32
35	1.16	252	38.8	4.95	46.5	12.3	3.97	1.09	0.980	8.04	0.396	1.13	412	5.50
31.8	1.24	239	36.7	5.05	43.4	11.4	3.79	1.10	1.00	7.69	0.360	0.937	380	5.64
50	0.741	269	44.9	4.28	56.5	17.4	5.64	1.09	1.05	10.9	0.613	3.23	411	4.77
45	0.845	251	41.9	4.36	52.0	15.8	5.30	1.09	1.04	10.1	0.550	2.33	373	4.88
40	0.952	234	39.0	4.46	47.7	14.2	4.98	1.10	1.04	9.31	0.490	1.69	336	5.01
35	1.07	216	36.0	4.59	43.2	12.6	4.64	1.11	1.05	8.62	0.428	1.24	297	5.18
31	1.17	202	33.7	4.71	39.7	11.3	4.37	1.11	1.08	8.15	0.425	1.00	267	5.34
10.6	0.284	55.3	9.22	4.22	11.6	0.378	0.307	0.349	0.269	0.635	0.129	0.0596	11.7	4.27
41.1	0.864	157	31.5	3.61	39.3	15.7	4.85	1.14	1.09	9.49	0.604	2.26	269	4.26
33.6	1.06	139	27.8	3.75	33.7	13.1	4.35	1.15	1.09	8.28	0.494	1.20	224	4.47
28.5	1.21	126	25.3	3.89	30.0	11.3	3.99	1.16	1.12	7.59	0.419	0.791	193	4.68
25	1.03	110	22.0	3.87	26.2	7.25	2.96	0.993	0.953	5.65	0.367	0.638	124	4.46
22	1.12	102	20.5	3.99	23.9	6.40	2.75	0.997	0.990	5.29	0.467	0.510	110	4.62
8.4	0.332	31.9	6.39	3.61	7.92	0.326	0.268	0.364	0.284	0.548	0.123	0.0413	7.00	3.68
6.5	0.182	22.9	4.59	3.43	5.90	0.133	0.137	0.262	0.194	0.284	0.0975	0.0191	2.76	3.46
25.4	0.986	87.9	19.5	3.43	23.5	7.57	2.99	1.01	0.970	5.70	0.415	0.691	104	4.08
23.9	1.04	84.9	18.9	3.48	22.5	7.14	2.89	1.01	0.981	5.51	0.390	0.599	98.0	4.15
22.8	1.04	63.8	15.9	3.09	19.1	7.01	2.81	1.02	1.01	5.37	0.419	0.572	75.2	3.84
21.4	1.09	61.5	15.4	3.13	18.2	6.58	2.71	1.02	1.02	5.18	0.452	0.495	70.8	3.91
20	0.843	54.4	13.6	3.04	16.4	4.42	2.02	0.867	0.840	3.86	0.367	0.441	47.8	3.58
18.7	0.889	52.4	13.1	3.09	15.6	4.15	1.95	0.868	0.849	3.72	0.344	0.380	45.0	3.65
8.5	0.542	23.3	5.82	3.05	6.95	0.624	0.431	0.500	0.428	0.875	0.156	0.0587	8.21	3.24

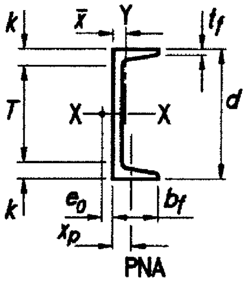


Table 1-6 (continued)
MC Shapes
Dimensions

Shape	Area, A	Depth, d		Web			Flange				Distance			r_{ts}	h_o
				Thickness, t_w		$\frac{t_w}{2}$	Width, b_f		Average Thickness, t_f		k	T	Workable Gage		
	in.^{2}	in.		in.		in.	in.		in.		in.	in.	in.	in.	in.
MC7×22.7	6.67	7.00	7	0.503	1/2	1/4	3.60	3 5/8	0.500	1/2	1 1/8	4 3/4	2[g]	1.23	6.50
×19.1	5.61	7.00	7	0.352	3/8	3/16	3.45	3 1/2	0.500	1/2	1 1/8	4 3/4	2[g]	1.18	6.50
MC6×18	5.29	6.00	6	0.379	3/8	3/16	3.50	3 1/2	0.475	1/2	1 1/16	3 7/8	2[g]	1.20	5.53
×15.3	4.49	6.00	6	0.340	5/16	3/16	3.50	3 1/2	0.385	3/8	7/8	4 1/4	2[g]	1.20	5.62
MC6×16.3	4.79	6.00	6	0.375	3/8	3/16	3.00	3	0.475	1/2	11/16	3 7/8	1 3/4[g]	1.03	5.53
×15.1	4.44	6.00	6	0.316	5/16	3/16	2.94	3	0.475	1/2	11/16	3 7/8	1 3/4[g]	1.01	5.53
MC6×12	3.53	6.00	6	0.310	5/16	3/16	2.50	2 1/2	0.375	3/8	7/8	4 1/4	1 1/2[g]	0.856	5.63
MC6×7	2.09	6.00	6	0.179	3/16	1/8	1.88	1 7/8	0.291	5/16	3/4	4 1/2	—	0.638	5.71
×6.5	1.95	6.00	6	0.155	1/8	1/16	1.85	1 7/8	0.291	5/16	3/4	4 1/2	—	0.630	5.71
MC4×13.8	4.03	4.00	4	0.500	1/2	1/4	2.50	2 1/2	0.500	1/2	1	2	—	0.852	3.50
MC3×7.1	2.11	3.00	3	0.312	5/16	3/16	1.94	2	0.351	3/8	13/16	1 3/8	—	0.657	2.65

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

— Flange is too narrow to establish a workable gage.

Table 1–6 (continued)
MC Shapes
Properties

MC7–MC3

Nom-inal Wt.	Shear Ctr, e_o	Axis X-X				Axis Y-Y						Torsional Properties		
		I	S	r	Z	I	S	r	$\bar{x}$	Z	x_p	J	C_w	$\bar{r}_o$
lb/ft	in.	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
22.7	1.01	47.4	13.5	2.67	16.4	7.24	2.83	1.04	1.04	5.38	0.477	0.625	58.3	3.53
19.1	1.15	43.1	12.3	2.77	14.5	6.06	2.55	1.04	1.08	4.85	0.579	0.407	49.3	3.70
18	1.17	29.7	9.89	2.37	11.7	5.88	2.47	1.05	1.12	4.68	0.644	0.379	34.6	3.46
15.3	1.16	25.3	8.44	2.38	9.91	4.91	2.01	1.05	1.05	3.85	0.511	0.223	30.0	3.41
16.3	0.930	26.0	8.66	2.33	10.4	3.77	1.82	0.887	0.927	3.47	0.465	0.336	22.1	3.11
15.1	0.982	24.9	8.30	2.37	9.83	3.46	1.73	0.883	0.940	3.30	0.543	0.285	20.5	3.18
12	0.725	18.7	6.24	2.30	7.47	1.85	1.03	0.724	0.704	1.97	0.294	0.155	11.3	2.80
7	0.583	11.4	3.81	2.34	4.50	0.603	0.439	0.537	0.501	0.865	0.174	0.0464	4.00	2.63
6.5	0.612	11.0	3.66	2.38	4.28	0.565	0.422	0.539	0.513	0.836	0.191	0.0412	3.75	2.68
13.8	0.643	8.85	4.43	1.48	5.53	2.13	1.29	0.727	0.849	2.40	0.508	0.373	4.84	2.23
7.1	0.574	2.72	1.81	1.14	2.24	0.666	0.518	0.562	0.653	0.998	0.414	0.0928	0.915	1.76

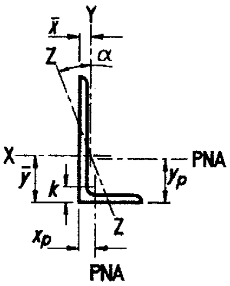


Table 1-7
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L8×8×1 1/8	1 3/4	56.9	16.7	98.1	17.5	2.41	2.40	31.6	1.05	7.13	32.5	4.29
×1	1 5/8	51.0	15.0	89.1	15.8	2.43	2.36	28.5	0.943	5.08	23.4	4.32
×7/8	1 1/2	45.0	13.2	79.7	14.0	2.45	2.31	25.3	0.832	3.46	16.1	4.36
×3/4	1 3/8	38.9	11.4	69.9	12.2	2.46	2.26	22.0	0.720	2.21	10.4	4.39
×5/8	1 1/4	32.7	9.61	59.6	10.3	2.48	2.21	18.6	0.606	1.30	6.16	4.42
×9/16	1 3/16	29.6	8.68	54.2	9.33	2.49	2.19	16.8	0.548	0.961	4.55	4.43
×1/2	1 1/8	26.4	7.75	48.8	8.36	2.49	2.17	15.1	0.490	0.683	3.23	4.45
L8×6×1	1 1/2	44.2	13.0	80.9	15.1	2.49	2.65	27.3	1.47	4.34	16.3	3.88
×7/8	1 3/8	39.1	11.5	72.4	13.4	2.50	2.60	24.3	1.41	2.96	11.3	3.92
×3/4	1 1/4	33.8	9.94	63.5	11.7	2.52	2.55	21.1	1.34	1.90	7.28	3.95
×5/8	1 1/8	28.5	8.36	54.2	9.86	2.54	2.50	17.9	1.27	1.12	4.33	3.98
×9/16	1 1/16	25.7	7.56	49.4	8.94	2.55	2.48	16.2	1.23	0.823	3.20	3.99
×1/2	1	23.0	6.75	44.4	8.01	2.55	2.46	14.6	1.20	0.584	2.28	4.01
×7/16	15/16	20.2	5.93	39.3	7.06	2.56	2.43	12.9	1.16	0.396	1.55	4.02
L8×4×1	1 1/2	37.4	11.0	69.7	14.0	2.51	3.03	24.3	2.47	3.68	12.9	3.75
×7/8	1 3/8	33.1	9.73	62.6	12.5	2.53	2.99	21.7	2.41	2.51	8.89	3.78
×3/4	1 1/4	28.7	8.44	55.0	10.9	2.55	2.94	18.9	2.34	1.61	5.75	3.80
×5/8	1 1/8	24.2	7.11	47.0	9.20	2.56	2.89	16.1	2.27	0.955	3.42	3.83
×9/16	1 1/16	21.9	6.43	42.9	8.34	2.57	2.86	14.6	2.23	0.704	2.53	3.84
×1/2	1	19.6	5.75	38.6	7.48	2.58	2.84	13.1	2.20	0.501	1.80	3.86
×7/16	15/16	17.2	5.06	34.2	6.59	2.59	2.81	11.6	2.16	0.340	1.22	3.87
L7×4×3/4	1 1/4	26.2	7.69	37.8	8.39	2.21	2.50	14.8	1.87	1.47	3.97	3.31
×5/8	1 1/8	22.1	6.48	32.4	7.12	2.23	2.45	12.5	1.80	0.868	2.37	3.34
×1/2	1	17.9	5.25	26.6	5.79	2.25	2.40	10.2	1.74	0.456	1.25	3.37
×7/16	15/16	15.7	4.62	23.6	5.11	2.26	2.38	9.03	1.70	0.310	0.851	3.38
×3/8	7/8	13.6	3.98	20.5	4.42	2.27	2.35	7.81	1.67	0.198	0.544	3.40
L6×6×1	1 1/2	37.4	11.0	35.4	8.55	1.79	1.86	15.4	0.918	3.68	9.24	3.18
×7/8	1 3/8	33.1	9.75	31.9	7.61	1.81	1.81	13.7	0.813	2.51	6.41	3.21
×3/4	1 1/4	28.7	8.46	28.1	6.64	1.82	1.77	11.9	0.705	1.61	4.17	3.24
×5/8	1 1/8	24.2	7.13	24.1	5.64	1.84	1.72	10.1	0.594	0.955	2.50	3.28
×9/16	1 1/16	21.9	6.45	22.0	5.12	1.85	1.70	9.18	0.538	0.704	1.85	3.29
×1/2	1	19.6	5.77	19.9	4.59	1.86	1.67	8.22	0.481	0.501	1.32	3.31
×7/16	15/16	17.2	5.08	17.6	4.06	1.86	1.65	7.25	0.423	0.340	0.899	3.32
×3/8	7/8	14.9	4.38	15.4	3.51	1.87	1.62	6.27	0.365	0.218	0.575	3.34
×5/16	13/16	12.4	3.67	13.0	2.95	1.88	1.60	5.26	0.306	0.129	0.338	3.35

Note: For compactness criteria, refer to the end of Table 1-7.

Table 1-7 (continued)
Angles
Properties

L8-L6

Shape	Axis Y-Y						Axis Z-Z				Q_s
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	Tan α	F_y = 36 ksi
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.		
L8×8×1 1/8	98.1	17.5	2.41	2.40	31.6	1.05	40.9	7.23	1.56	1.00	1.00
×1	89.1	15.8	2.43	2.36	28.5	0.943	36.8	6.51	1.56	1.00	1.00
×7/8	79.7	14.0	2.45	2.31	25.3	0.832	32.7	5.78	1.57	1.00	1.00
×3/4	69.9	12.2	2.46	2.26	22.0	0.720	28.5	5.04	1.57	1.00	1.00
×5/8	59.6	10.3	2.48	2.21	18.6	0.606	24.2	4.27	1.58	1.00	0.997
×9/16	54.2	9.33	2.49	2.19	16.8	0.548	22.0	3.88	1.58	1.00	0.959
×1/2	48.8	8.36	2.49	2.17	15.1	0.490	19.7	3.49	1.59	1.00	0.912
L8×6×1	38.8	8.92	1.72	1.65	16.2	0.816	21.3	4.84	1.28	0.542	1.00
×7/8	34.9	7.94	1.74	1.60	14.4	0.721	18.9	4.31	1.28	0.546	1.00
×3/4	30.8	6.92	1.75	1.56	12.5	0.624	16.5	3.78	1.29	0.550	1.00
×5/8	26.4	5.88	1.77	1.51	10.5	0.526	14.1	3.22	1.29	0.554	0.997
×9/16	24.1	5.34	1.78	1.49	9.52	0.476	12.8	2.94	1.30	0.556	0.959
×1/2	21.7	4.79	1.79	1.46	8.52	0.425	11.5	2.64	1.30	0.557	0.912
×7/16	19.3	4.23	1.80	1.44	7.50	0.374	10.2	2.35	1.31	0.559	0.850
L8×4×1	11.6	3.94	1.03	1.04	7.73	0.691	7.87	2.15	0.844	0.247	1.00
×7/8	10.5	3.51	1.04	0.997	6.77	0.612	7.01	1.93	0.846	0.252	1.00
×3/4	9.37	3.07	1.05	0.949	5.82	0.531	6.13	1.70	0.850	0.257	1.00
×5/8	8.11	2.62	1.06	0.902	4.86	0.448	5.24	1.47	0.856	0.262	0.997
×9/16	7.44	2.38	1.07	0.878	4.39	0.405	4.79	1.34	0.859	0.264	0.959
×1/2	6.75	2.15	1.08	0.854	3.91	0.363	4.32	1.22	0.863	0.266	0.912
×7/16	6.03	1.90	1.09	0.829	3.42	0.320	3.84	1.09	0.867	0.268	0.850
L7×4×3/4	9.00	3.01	1.08	1.00	5.60	0.550	5.64	1.71	0.855	0.324	1.00
×5/8	7.79	2.56	1.10	0.958	4.69	0.464	4.80	1.47	0.860	0.329	1.00
×1/2	6.48	2.10	1.11	0.910	3.77	0.376	3.95	1.21	0.866	0.334	0.965
×7/16	5.79	1.86	1.12	0.886	3.31	0.331	3.50	1.08	0.869	0.337	0.912
×3/8	5.06	1.61	1.12	0.861	2.84	0.286	3.05	0.942	0.873	0.339	0.840
L6×6×1	35.4	8.55	1.79	1.86	15.4	0.918	15.0	3.53	1.17	1.00	1.00
×7/8	31.9	7.61	1.81	1.81	13.7	0.813	13.3	3.13	1.17	1.00	1.00
×3/4	28.1	6.64	1.82	1.77	11.9	0.705	11.6	2.73	1.17	1.00	1.00
×5/8	24.1	5.64	1.84	1.72	10.1	0.594	9.83	2.32	1.17	1.00	1.00
×9/16	22.0	5.12	1.85	1.70	9.17	0.538	8.94	2.11	1.18	1.00	1.00
×1/2	19.9	4.59	1.86	1.67	8.22	0.481	8.04	1.89	1.18	1.00	1.00
×7/16	17.6	4.06	1.86	1.65	7.25	0.423	7.11	1.68	1.18	1.00	0.973
×3/8	15.4	3.51	1.87	1.62	6.26	0.365	6.17	1.45	1.19	1.00	0.912
×5/16	13.0	2.95	1.88	1.60	5.26	0.306	5.20	1.23	1.19	1.00	0.826

Note: For compactness criteria, refer to the end of Table 1-7.

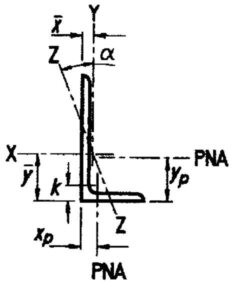


Table 1–7 (continued)
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L6×4×7/8	1 3/8	27.2	7.98	27.7	7.13	1.86	2.12	12.7	1.44	2.03	4.04	2.82
×3/4	1 1/4	23.6	6.94	24.5	6.23	1.88	2.07	11.1	1.38	1.31	2.64	2.85
×5/8	1 1/8	20.0	5.86	21.0	5.29	1.89	2.03	9.44	1.31	0.775	1.59	2.88
×9/16	1 1/16	18.1	5.31	19.2	4.81	1.90	2.00	8.59	1.28	0.572	1.18	2.90
×1/2	1	16.2	4.75	17.3	4.31	1.91	1.98	7.71	1.25	0.407	0.843	2.91
×7/16	15/16	14.3	4.18	15.4	3.81	1.92	1.95	6.81	1.22	0.276	0.575	2.93
×3/8	7/8	12.3	3.61	13.4	3.30	1.93	1.93	5.89	1.19	0.177	0.369	2.94
×5/16	13/16	10.3	3.03	11.4	2.77	1.94	1.90	4.96	1.16	0.104	0.217	2.96
L6×3 1/2×1/2	1	15.3	4.50	16.6	4.23	1.92	2.07	7.49	1.48	0.386	0.779	2.88
×3/8	7/8	11.7	3.42	12.9	3.23	1.93	2.02	5.74	1.41	0.168	0.341	2.90
×5/16	13/16	9.80	2.87	10.9	2.72	1.94	2.00	4.84	1.38	0.0990	0.201	2.92
L5×5×7/8	1 3/8	27.2	7.98	17.8	5.16	1.49	1.56	9.31	0.802	2.07	3.53	2.64
×3/4	1 1/4	23.6	6.94	15.7	4.52	1.50	1.52	8.14	0.698	1.33	2.32	2.67
×5/8	1 1/8	20.0	5.86	13.6	3.85	1.52	1.47	6.93	0.590	0.792	1.40	2.70
×1/2	1	16.2	4.75	11.3	3.15	1.53	1.42	5.66	0.479	0.417	0.744	2.73
×7/16	15/16	14.3	4.18	10.0	2.78	1.54	1.40	5.00	0.422	0.284	0.508	2.74
×3/8	7/8	12.3	3.61	8.76	2.41	1.55	1.37	4.33	0.365	0.183	0.327	2.76
×5/16	13/16	10.3	3.03	7.44	2.04	1.56	1.35	3.65	0.307	0.108	0.193	2.77
L5×3 1/2×3/4	1 3/16	19.8	5.81	13.9	4.26	1.55	1.74	7.60	1.12	1.09	1.52	2.36
×5/8	1 1/16	16.8	4.92	12.0	3.63	1.56	1.69	6.50	1.06	0.651	0.918	2.39
×1/2	15/16	13.6	4.00	9.96	2.97	1.58	1.65	5.33	0.997	0.343	0.491	2.42
×3/8	13/16	10.4	3.05	7.75	2.28	1.59	1.60	4.09	0.933	0.150	0.217	2.45
×5/16	3/4	8.70	2.56	6.58	1.92	1.60	1.57	3.45	0.901	0.0883	0.128	2.47
×1/4	11/16	7.00	2.06	5.36	1.55	1.61	1.55	2.78	0.868	0.0464	0.0670	2.48
L5×3×1/2	15/16	12.8	3.75	9.43	2.89	1.58	1.74	5.12	1.25	0.322	0.444	2.38
×7/16	7/8	11.3	3.31	8.41	2.56	1.59	1.72	4.53	1.21	0.220	0.304	2.39
×3/8	13/16	9.80	2.86	7.35	2.22	1.60	1.69	3.93	1.18	0.141	0.196	2.41
×5/16	3/4	8.20	2.40	6.24	1.87	1.61	1.67	3.32	1.15	0.0832	0.116	2.42
×1/4	11/16	6.60	1.94	5.09	1.51	1.62	1.64	2.68	1.12	0.0438	0.0606	2.43
L4×4×3/4	1 1/8	18.5	5.44	7.62	2.79	1.18	1.27	5.02	0.679	1.02	1.12	2.10
×5/8	1	15.7	4.61	6.62	2.38	1.20	1.22	4.28	0.576	0.610	0.680	2.13
×1/2	7/8	12.8	3.75	5.52	1.96	1.21	1.18	3.50	0.468	0.322	0.366	2.16
×7/16	13/16	11.3	3.31	4.93	1.73	1.22	1.15	3.10	0.413	0.220	0.252	2.18
×3/8	3/4	9.80	2.86	4.32	1.50	1.23	1.13	2.69	0.357	0.141	0.162	2.19
×5/16	11/16	8.20	2.40	3.67	1.27	1.24	1.11	2.26	0.300	0.0832	0.0963	2.21
×1/4	5/8	6.60	1.94	3.00	1.03	1.25	1.08	1.82	0.242	0.0438	0.0505	2.22

Note: For compactness criteria, refer to the end of Table 1–7.

Table 1–7 (continued)
Angles
Properties

L6-L4

Shape	Axis Y-Y						Axis Z-Z				Q_s
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	Tan α	F_y = 36 ksi
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.		
L6×4×7/8	9.70	3.37	1.10	1.12	6.26	0.665	5.82	1.90	0.854	0.421	1.00
×3/4	8.63	2.95	1.12	1.07	5.42	0.578	5.08	1.66	0.856	0.428	1.00
×5/8	7.48	2.52	1.13	1.03	4.56	0.488	4.32	1.42	0.859	0.435	1.00
×9/16	6.86	2.29	1.14	1.00	4.13	0.442	3.94	1.30	0.861	0.438	1.00
×1/2	6.22	2.06	1.14	0.981	3.69	0.396	3.55	1.17	0.864	0.440	1.00
×7/16	5.56	1.83	1.15	0.957	3.24	0.349	3.14	1.04	0.867	0.443	0.973
×3/8	4.86	1.58	1.16	0.933	2.79	0.301	2.73	0.908	0.870	0.446	0.912
×5/16	4.13	1.34	1.17	0.908	2.33	0.252	2.31	0.769	0.874	0.449	0.826
L6×3 1/2×1/2	4.24	1.59	0.968	0.829	2.88	0.376	2.58	0.914	0.756	0.343	1.00
×3/8	3.33	1.22	0.984	0.781	2.18	0.287	2.00	0.714	0.763	0.349	0.912
×5/16	2.84	1.03	0.991	0.756	1.82	0.241	1.70	0.609	0.767	0.352	0.826
L5×5×7/8	17.8	5.16	1.49	1.56	9.30	0.802	7.56	2.14	0.971	1.00	1.00
×3/4	15.7	4.52	1.50	1.52	8.14	0.698	6.59	1.86	0.972	1.00	1.00
×5/8	13.6	3.85	1.52	1.47	6.92	0.590	5.61	1.59	0.975	1.00	1.00
×1/2	11.3	3.15	1.53	1.42	5.66	0.479	4.60	1.30	0.980	1.00	1.00
×7/16	10.0	2.78	1.54	1.40	5.00	0.422	4.08	1.15	0.983	1.00	1.00
×3/8	8.76	2.41	1.55	1.37	4.33	0.365	3.55	1.00	0.986	1.00	0.983
×5/16	7.44	2.04	1.56	1.35	3.65	0.307	3.01	0.850	0.990	1.00	0.912
L5×3 1/2×3/4	5.52	2.20	0.974	0.993	4.07	0.582	3.22	1.22	0.744	0.464	1.00
×5/8	4.80	1.88	0.987	0.947	3.43	0.493	2.74	1.05	0.746	0.472	1.00
×1/2	4.02	1.55	1.00	0.901	2.79	0.400	2.25	0.862	0.750	0.479	1.00
×3/8	3.15	1.19	1.02	0.854	2.12	0.305	1.74	0.670	0.755	0.485	0.983
×5/16	2.69	1.01	1.02	0.829	1.77	0.256	1.47	0.569	0.758	0.489	0.912
×1/4	2.20	0.816	1.03	0.804	1.42	0.207	1.19	0.463	0.761	0.491	0.804
L5×3×1/2	2.55	1.13	0.824	0.746	2.08	0.375	1.55	0.645	0.642	0.357	1.00
×7/16	2.29	1.00	0.831	0.722	1.82	0.331	1.37	0.575	0.644	0.361	1.00
×3/8	2.01	0.874	0.838	0.698	1.57	0.286	1.20	0.503	0.646	0.364	0.983
×5/16	1.72	0.739	0.846	0.673	1.31	0.241	1.01	0.428	0.649	0.368	0.912
×1/4	1.41	0.600	0.853	0.648	1.05	0.194	0.825	0.350	0.652	0.371	0.804
L4×4×3/4	7.62	2.79	1.18	1.27	5.01	0.679	3.25	1.15	0.774	1.00	1.00
×5/8	6.62	2.38	1.20	1.22	4.28	0.576	2.76	0.975	0.774	1.00	1.00
×1/2	5.52	1.96	1.21	1.18	3.50	0.468	2.25	0.797	0.776	1.00	1.00
×7/16	4.93	1.73	1.22	1.15	3.10	0.413	2.00	0.706	0.777	1.00	1.00
×3/8	4.32	1.50	1.23	1.13	2.68	0.357	1.73	0.613	0.779	1.00	1.00
×5/16	3.67	1.27	1.24	1.11	2.26	0.300	1.46	0.517	0.781	1.00	0.997
×1/4	3.00	1.03	1.25	1.08	1.82	0.242	1.18	0.419	0.783	1.00	0.912

Note: For compactness criteria, refer to the end of Table 1–7.

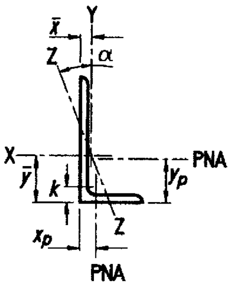


Table 1-7 (continued)
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L4×3½×½	7/8	11.9	3.50	5.30	1.92	1.23	1.24	3.46	0.497	0.301	0.302	2.03
×3/8	3/4	9.10	2.67	4.15	1.48	1.25	1.20	2.66	0.433	0.132	0.134	2.06
×5/16	11/16	7.70	2.25	3.53	1.25	1.25	1.17	2.24	0.401	0.0782	0.0798	2.08
×1/4	5/8	6.20	1.81	2.89	1.01	1.26	1.14	1.81	0.368	0.0412	0.0419	2.09
L4×3×5/8	1	13.6	3.89	6.01	2.28	1.23	1.37	4.08	0.810	0.529	0.472	1.91
×1/2	7/8	11.1	3.25	5.02	1.87	1.24	1.32	3.36	0.747	0.281	0.255	1.94
×3/8	3/4	8.50	2.48	3.94	1.44	1.26	1.27	2.60	0.683	0.123	0.114	1.97
×5/16	11/16	7.20	2.09	3.36	1.22	1.27	1.25	2.19	0.651	0.0731	0.0676	1.98
×1/4	5/8	5.80	1.69	2.75	0.988	1.27	1.22	1.77	0.618	0.0386	0.0356	1.99
L3½×3½×½	7/8	11.1	3.25	3.63	1.48	1.05	1.05	2.66	0.466	0.281	0.238	1.87
×7/16	13/16	9.80	2.87	3.25	1.32	1.06	1.03	2.36	0.412	0.192	0.164	1.89
×3/8	3/4	8.50	2.48	2.86	1.15	1.07	1.00	2.06	0.357	0.123	0.106	1.90
×5/16	11/16	7.20	2.09	2.44	0.969	1.08	0.979	1.74	0.301	0.0731	0.0634	1.92
×1/4	5/8	5.80	1.69	2.00	0.787	1.09	0.954	1.41	0.243	0.0386	0.0334	1.93
L3½×3×½	7/8	10.2	3.00	3.45	1.45	1.07	1.12	2.61	0.480	0.260	0.191	1.75
×7/16	13/16	9.10	2.65	3.10	1.29	1.08	1.09	2.32	0.446	0.178	0.132	1.76
×3/8	3/4	7.90	2.30	2.73	1.12	1.09	1.07	2.03	0.411	0.114	0.0858	1.78
×5/16	11/16	6.60	1.93	2.33	0.951	1.09	1.05	1.72	0.375	0.0680	0.0512	1.79
×1/4	5/8	5.40	1.56	1.92	0.773	1.10	1.02	1.39	0.336	0.0360	0.0270	1.80
L3½×2½×½	7/8	9.40	2.75	3.24	1.41	1.08	1.20	2.52	0.736	0.234	0.159	1.66
×3/8	3/4	7.20	2.11	2.56	1.09	1.10	1.15	1.96	0.668	0.103	0.0714	1.69
×5/16	11/16	6.10	1.78	2.20	0.925	1.11	1.13	1.67	0.633	0.0611	0.0426	1.71
×1/4	5/8	4.90	1.44	1.81	0.753	1.12	1.10	1.36	0.596	0.0322	0.0225	1.72
L3×3×½	7/8	9.40	2.75	2.20	1.06	0.895	0.929	1.91	0.458	0.230	0.144	1.59
×7/16	13/16	8.30	2.43	1.98	0.946	0.903	0.907	1.70	0.405	0.157	0.100	1.60
×3/8	3/4	7.20	2.11	1.75	0.825	0.910	0.884	1.48	0.351	0.101	0.0652	1.62
×5/16	11/16	6.10	1.78	1.50	0.699	0.918	0.860	1.26	0.296	0.0597	0.0390	1.64
×1/4	5/8	4.90	1.44	1.23	0.569	0.926	0.836	1.02	0.239	0.0313	0.0206	1.65
×3/16	9/16	3.71	1.09	0.948	0.433	0.933	0.812	0.774	0.181	0.0136	0.00899	1.67
L3×2½×½	7/8	8.50	2.50	2.07	1.03	0.910	0.995	1.86	0.494	0.213	0.112	1.46
×7/16	13/16	7.60	2.21	1.87	0.921	0.917	0.972	1.66	0.462	0.146	0.0777	1.48
×3/8	3/4	6.60	1.92	1.65	0.803	0.924	0.949	1.45	0.430	0.0943	0.0507	1.49
×5/16	11/16	5.60	1.67	1.41	0.681	0.932	0.925	1.23	0.397	0.0560	0.0304	1.51
×1/4	5/8	4.50	1.31	1.16	0.555	0.940	0.900	1.000	0.363	0.0296	0.0161	1.52
×3/16	9/16	3.39	0.996	0.899	0.423	0.947	0.874	0.761	0.328	0.0130	0.00705	1.54

Note: For compactness criteria, refer to the end of Table 1–7.

Table 1–7 (continued)
Angles
Properties

L4-L3

Shape	Axis Y-Y						Axis Z-Z				Q_s
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	Tan α	F_y = 36 ksi
	in.^4	in.^3	in.	in.	in.^3	in.	in.^4	in.^3	in.		
L4×3½×½	3.76	1.50	1.04	0.994	2.69	0.438	1.80	0.719	0.716	0.750	1.00
×3/8	2.96	1.16	1.05	0.947	2.06	0.334	1.38	0.555	0.719	0.755	1.00
×5/16	2.52	0.980	1.06	0.923	1.74	0.281	1.17	0.470	0.721	0.757	0.997
×¼	2.07	0.794	1.07	0.897	1.40	0.227	0.950	0.382	0.723	0.759	0.912
L4×3×5/8	2.85	1.34	0.845	0.867	2.45	0.498	1.59	0.720	0.631	0.534	1.00
×½	2.40	1.10	0.858	0.822	1.99	0.407	1.30	0.592	0.633	0.542	1.00
×3/8	1.89	0.851	0.873	0.775	1.52	0.311	1.01	0.460	0.636	0.551	1.00
×5/16	1.62	0.721	0.880	0.750	1.28	0.262	0.851	0.390	0.638	0.554	0.997
×¼	1.33	0.585	0.887	0.725	1.03	0.211	0.691	0.318	0.639	0.558	0.912
L3½×3½×½	3.63	1.48	1.05	1.05	2.66	0.466	1.51	0.609	0.679	1.00	1.00
×7/16	3.25	1.32	1.06	1.03	2.36	0.412	1.34	0.540	0.681	1.00	1.00
×3/8	2.86	1.15	1.07	1.00	2.05	0.357	1.17	0.471	0.683	1.00	1.00
×5/16	2.44	0.969	1.08	0.979	1.74	0.301	0.989	0.400	0.685	1.00	1.00
×¼	2.00	0.787	1.09	0.954	1.41	0.243	0.807	0.326	0.688	1.00	0.965
L3½×3×½	2.32	1.09	0.877	0.869	1.97	0.431	1.15	0.537	0.618	0.713	1.00
×7/16	2.09	0.971	0.885	0.846	1.75	0.382	1.03	0.478	0.620	0.717	1.00
×3/8	1.84	0.847	0.892	0.823	1.52	0.331	0.895	0.418	0.622	0.720	1.00
×5/16	1.58	0.718	0.900	0.798	1.28	0.279	0.761	0.356	0.624	0.722	1.00
×¼	1.30	0.585	0.908	0.773	1.04	0.226	0.623	0.292	0.628	0.725	0.965
L3½×2½×½	1.36	0.756	0.701	0.701	1.39	0.395	0.782	0.420	0.532	0.485	1.00
×3/8	1.09	0.589	0.716	0.655	1.07	0.303	0.608	0.329	0.535	0.495	1.00
×5/16	0.937	0.501	0.723	0.632	0.900	0.256	0.518	0.281	0.538	0.500	1.00
×¼	0.775	0.410	0.731	0.607	0.728	0.207	0.425	0.232	0.541	0.504	0.965
L3×3×½	2.20	1.06	0.895	0.929	1.91	0.458	0.924	0.436	0.580	1.00	1.00
×7/16	1.98	0.946	0.903	0.907	1.70	0.405	0.819	0.386	0.580	1.00	1.00
×3/8	1.75	0.825	0.910	0.884	1.48	0.351	0.712	0.336	0.581	1.00	1.00
×5/16	1.50	0.699	0.918	0.860	1.25	0.296	0.603	0.284	0.583	1.00	1.00
×¼	1.23	0.569	0.926	0.836	1.02	0.239	0.491	0.231	0.585	1.00	1.00
×3/16	0.948	0.433	0.933	0.812	0.774	0.181	0.374	0.176	0.586	1.00	0.912
L3×2½×½	1.29	0.736	0.718	0.746	1.34	0.418	0.666	0.370	0.516	0.666	1.00
×7/16	1.17	0.656	0.724	0.724	1.19	0.370	0.591	0.329	0.516	0.671	1.00
×3/8	1.03	0.573	0.731	0.701	1.03	0.321	0.514	0.287	0.517	0.675	1.00
×5/16	0.888	0.487	0.739	0.677	0.873	0.271	0.437	0.244	0.518	0.679	1.00
×¼	0.734	0.397	0.746	0.653	0.707	0.220	0.356	0.199	0.520	0.683	1.00
×3/16	0.568	0.303	0.753	0.627	0.536	0.167	0.272	0.153	0.521	0.687	0.912

Note: For compactness criteria, refer to the end of Table 1–7.

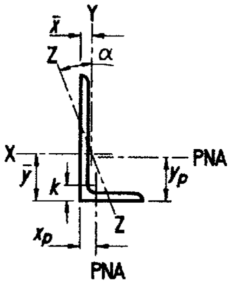
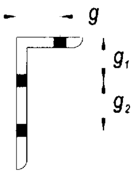


Table 1–7 (continued)
Angles
Properties

Shape	k	Wt.	Area, A	Axis X-X						Flexural-Torsional Properties		
				I	S	r	$\bar{y}$	Z	y_p	J	C_w	$\bar{r}_o$
	in.	lb/ft	in.^{2}	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{6}	in.
L3×2×½	13/16	7.70	2.25	1.92	1.00	0.922	1.08	1.78	0.736	0.192	0.0908	1.39
×⅜	11/16	5.90	1.73	1.54	0.779	0.937	1.03	1.39	0.668	0.0855	0.0413	1.42
×5/16	⅝	5.00	1.46	1.32	0.662	0.945	1.01	1.19	0.633	0.0510	0.0248	1.43
×¼	9/16	4.10	1.19	1.09	0.541	0.953	0.980	0.969	0.596	0.0270	0.0132	1.45
×3/16	½	3.07	0.902	0.847	0.414	0.961	0.952	0.743	0.556	0.0119	0.00576	1.46
L2½×2½×½	¾	7.70	2.25	1.22	0.716	0.735	0.803	1.29	0.450	0.188	0.0791	1.30
×⅜	⅝	5.90	1.73	0.972	0.558	0.749	0.758	1.01	0.347	0.0833	0.0362	1.33
×5/16	9/16	5.00	1.46	0.837	0.474	0.756	0.735	0.853	0.293	0.0495	0.0218	1.35
×¼	½	4.10	1.19	0.692	0.387	0.764	0.711	0.695	0.237	0.0261	0.0116	1.36
×3/16	7/16	3.07	0.900	0.535	0.295	0.771	0.687	0.529	0.180	0.0114	0.00510	1.38
L2½×2×⅜	⅝	5.30	1.55	0.914	0.546	0.766	0.826	0.982	0.425	0.0746	0.0268	1.22
×5/16	9/16	4.50	1.31	0.790	0.465	0.774	0.803	0.839	0.391	0.0444	0.0162	1.23
×¼	½	3.62	1.06	0.656	0.381	0.782	0.779	0.688	0.356	0.0235	0.00868	1.25
×3/16	7/16	2.75	0.809	0.511	0.293	0.790	0.754	0.529	0.318	0.0103	0.00382	1.26
L2½×1½×¼	½	3.22	0.938	0.594	0.364	0.792	0.866	0.644	0.606	0.0209	0.00694	1.19
×3/16	7/16	2.47	0.715	0.464	0.280	0.801	0.839	0.497	0.568	0.00921	0.00306	1.20
L2×2×⅜	⅝	4.70	1.36	0.476	0.348	0.591	0.632	0.629	0.342	0.0658	0.0174	1.05
×5/16	9/16	3.92	1.15	0.414	0.298	0.598	0.609	0.537	0.290	0.0393	0.0106	1.06
×¼	½	3.19	0.938	0.346	0.244	0.605	0.586	0.440	0.236	0.0209	0.00572	1.08
×3/16	7/16	2.44	0.715	0.271	0.188	0.612	0.561	0.338	0.180	0.00921	0.00254	1.09
×⅛	⅜	1.65	0.484	0.189	0.129	0.620	0.534	0.230	0.123	0.00293	0.000789	1.10



Workable Gages in Angle Legs, in.

Leg	8	7	6	5	4	3½	3	2½	2	1¾	1½	1⅜	1¼	1
g	4½	4	3½	3	2½	2	1¾	1⅜	1⅛	1	⅞	⅞	¾	⅝
g_1	3	2½	2¼	2										
g_2	3	3	2½	1¾										

Note: Other gages are permitted to suit specific requirements subject to clearances and edge distance limitations

Table 1–7 (continued)
Angles
Properties

L3-L2

Shape	Axis Y-Y						Axis Z-Z				Q_s
	I	S	r	$\bar{x}$	Z	x_p	I	S	r	Tan α	$F_y = 36$ ksi
	in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.		
L3×2×1/2	0.667	0.470	0.543	0.580	0.887	0.377	0.409	0.266	0.425	0.413	1.00
×3/8	0.539	0.368	0.555	0.535	0.679	0.291	0.318	0.209	0.426	0.426	1.00
×5/16	0.467	0.314	0.562	0.511	0.572	0.247	0.271	0.179	0.428	0.432	1.00
×1/4	0.390	0.258	0.569	0.487	0.463	0.200	0.223	0.149	0.431	0.437	1.00
×3/16	0.305	0.198	0.577	0.462	0.351	0.153	0.173	0.116	0.435	0.442	0.912
L2 1/2×2 1/2×1/2	1.22	0.716	0.735	0.803	1.29	0.450	0.521	0.295	0.481	1.00	1.00
×3/8	0.972	0.558	0.749	0.758	1.00	0.347	0.400	0.226	0.481	1.00	1.00
×5/16	0.837	0.474	0.756	0.735	0.853	0.293	0.339	0.192	0.481	1.00	1.00
×1/4	0.692	0.387	0.764	0.711	0.694	0.237	0.275	0.156	0.482	1.00	1.00
×3/16	0.535	0.295	0.771	0.687	0.528	0.180	0.210	0.119	0.482	1.00	0.983
L2 1/2×2×3/8	0.513	0.361	0.574	0.578	0.657	0.311	0.273	0.189	0.419	0.612	1.00
×5/16	0.446	0.309	0.581	0.555	0.557	0.264	0.233	0.161	0.420	0.618	1.00
×1/4	0.372	0.253	0.589	0.532	0.454	0.214	0.191	0.133	0.423	0.624	1.00
×3/16	0.292	0.195	0.597	0.508	0.347	0.164	0.149	0.104	0.426	0.628	0.983
L2 1/2×1 1/2×1/4	0.160	0.142	0.411	0.372	0.261	0.189	0.0975	0.0818	0.321	0.354	1.00
×3/16	0.126	0.110	0.418	0.347	0.198	0.145	0.0760	0.0644	0.324	0.360	0.983
L2×2×3/8	0.476	0.348	0.591	0.632	0.628	0.342	0.203	0.144	0.386	1.00	1.00
×5/16	0.414	0.298	0.598	0.609	0.536	0.290	0.173	0.122	0.386	1.00	1.00
×1/4	0.346	0.244	0.605	0.586	0.440	0.236	0.141	0.1000	0.387	1.00	1.00
×3/16	0.271	0.188	0.612	0.561	0.338	0.180	0.109	0.0771	0.389	1.00	1.00
×1/8	0.189	0.129	0.620	0.534	0.230	0.123	0.0751	0.0531	0.391	1.00	0.912

t	Compactness Criteria for Angles		
	Compression	Flexure	
	non-slender up to	compact up to	non-compact up to
	Width of angle leg, in.		
1 1/8	8	8	—
1	↓	↓	—
7/8	↓	↓	—
3/4	↓	↓	—
5/8	↓	↓	—
9/16	7	↓	—
1/2	6	7	8
7/16	5	6	↓
3/8	4	5	↓
5/16	4	4	↓
1/4	3	3 1/2	6
3/16	2	2 1/2	4
1/8	1 1/2	1 1/2	3

Note: Compactness criteria given for F_y = 36 ksi. C_v = 1.0 for all angles.

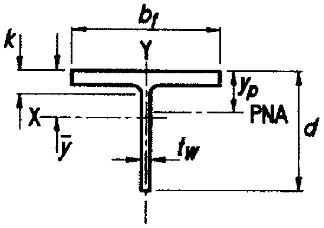


Table 1–8
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT22×167.5^{c}	49.2	22.0	22	1.03	1	$\frac{1}{2}$	22.6	15.9	16	1.77	$1\frac{3}{4}$	2.56	$2\frac{5}{8}$	$5\frac{1}{2}$
×145^{c}	42.7	21.8	$21\frac{3}{4}$	0.865	$\frac{7}{8}$	$\frac{7}{16}$	18.9	15.8	$15\frac{7}{8}$	1.58	$1\frac{9}{16}$	2.36	$2\frac{7}{16}$	↓
×131^{c}	38.4	21.7	$21\frac{5}{8}$	0.785	$\frac{13}{16}$	$\frac{7}{16}$	17.0	15.8	$15\frac{3}{4}$	1.42	$1\frac{7}{16}$	2.20	$2\frac{1}{4}$	↓
×115^{c,v}	33.8	21.5	$21\frac{1}{2}$	0.710	$\frac{11}{16}$	$\frac{3}{8}$	15.2	15.8	$15\frac{3}{4}$	1.22	$1\frac{1}{4}$	2.01	$2\frac{1}{16}$	↓
WT20×296.5^{h}	87.2	21.5	$21\frac{1}{2}$	1.79	$1\frac{13}{16}$	$\frac{15}{16}$	38.5	16.7	$16\frac{3}{4}$	3.23	$3\frac{1}{4}$	4.41	$4\frac{1}{2}$	$7\frac{1}{2}$
×251.5^{h}	73.9	21.0	21	1.54	$1\frac{9}{16}$	$\frac{13}{16}$	32.3	16.4	$16\frac{3}{8}$	2.76	$2\frac{3}{4}$	3.94	4	↓
×215.5^{h}	63.4	20.6	$20\frac{5}{8}$	1.34	$1\frac{5}{16}$	$\frac{11}{16}$	27.6	16.2	$16\frac{1}{4}$	2.36	$2\frac{3}{8}$	3.54	$3\frac{5}{8}$	↓
×198.5^{h}	58.4	20.5	$20\frac{1}{2}$	1.22	$1\frac{1}{4}$	$\frac{5}{8}$	25.0	16.1	$16\frac{1}{8}$	2.20	$2\frac{3}{16}$	3.38	$3\frac{1}{2}$	↓
×186^{h}	54.6	20.3	$20\frac{3}{8}$	1.16	$1\frac{3}{16}$	$\frac{5}{8}$	23.6	16.1	$16\frac{1}{8}$	2.05	$2\frac{1}{16}$	3.23	$3\frac{5}{16}$	↓
×181^{c,h}	53.3	20.3	$20\frac{1}{4}$	1.12	$1\frac{1}{8}$	$\frac{9}{16}$	22.7	16.0	16	2.01	2	3.19	$3\frac{1}{4}$	↓
×162^{c}	47.7	20.1	$20\frac{1}{8}$	1.00	1	$\frac{1}{2}$	20.1	15.9	$15\frac{7}{8}$	1.81	$1\frac{13}{16}$	2.99	$3\frac{1}{16}$	↓
×148.5^{c}	43.7	19.9	$19\frac{7}{8}$	0.930	$\frac{15}{16}$	$\frac{1}{2}$	18.5	15.8	$15\frac{7}{8}$	1.65	$1\frac{5}{8}$	2.83	$2\frac{15}{16}$	↓
×138.5^{c}	40.7	19.8	$19\frac{7}{8}$	0.830	$\frac{13}{16}$	$\frac{7}{16}$	16.5	15.8	$15\frac{7}{8}$	1.58	$1\frac{9}{16}$	2.76	$2\frac{7}{8}$	↓
×124.5^{c}	36.7	19.7	$19\frac{3}{4}$	0.750	$\frac{3}{4}$	$\frac{3}{8}$	14.8	15.8	$15\frac{3}{4}$	1.42	$1\frac{7}{16}$	2.60	$2\frac{11}{16}$	↓
×107.5^{c,v}	31.7	19.5	$19\frac{1}{2}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.7	15.8	$15\frac{3}{4}$	1.22	$1\frac{1}{4}$	2.40	$2\frac{1}{2}$	↓
×99.5^{c,v}	29.2	19.3	$19\frac{3}{8}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.6	15.8	$15\frac{3}{4}$	1.07	$1\frac{1}{16}$	2.25	$2\frac{5}{16}$	↓
WT20×196^{h}	57.6	20.8	$20\frac{3}{4}$	1.42	$1\frac{7}{16}$	$\frac{3}{4}$	29.4	12.4	$12\frac{3}{8}$	2.52	$2\frac{1}{2}$	3.70	$3\frac{13}{16}$	$7\frac{1}{2}$
×165.5^{h}	48.7	20.4	$20\frac{3}{8}$	1.22	$1\frac{1}{4}$	$\frac{5}{8}$	24.9	12.2	$12\frac{1}{8}$	2.13	$2\frac{1}{8}$	3.31	$3\frac{3}{8}$	↓
×163.5^{h}	48.0	20.4	$20\frac{3}{8}$	1.18	$1\frac{3}{16}$	$\frac{5}{8}$	24.1	12.1	$12\frac{1}{8}$	2.13	$2\frac{1}{8}$	3.31	$3\frac{3}{8}$	↓
×147^{c}	43.1	20.2	$20\frac{1}{4}$	1.06	$1\frac{1}{16}$	$\frac{9}{16}$	21.4	12.0	12	1.93	$1\frac{15}{16}$	3.11	$3\frac{3}{16}$	↓
×139^{c}	41.0	20.1	$20\frac{1}{8}$	1.03	1	$\frac{1}{2}$	20.6	12.0	12	1.81	$1\frac{13}{16}$	2.99	$3\frac{1}{16}$	↓
×132^{c}	38.8	20.0	20	0.960	$\frac{15}{16}$	$\frac{1}{2}$	19.2	11.9	$11\frac{7}{8}$	1.73	$1\frac{3}{4}$	2.91	3	↓
×117.5^{c}	34.5	19.8	$19\frac{7}{8}$	0.830	$\frac{13}{16}$	$\frac{7}{16}$	16.5	11.9	$11\frac{7}{8}$	1.58	$1\frac{9}{16}$	2.76	$2\frac{7}{8}$	↓
×105.5^{c}	31.0	19.7	$19\frac{5}{8}$	0.750	$\frac{3}{4}$	$\frac{3}{8}$	14.8	11.8	$11\frac{3}{4}$	1.42	$1\frac{7}{16}$	2.60	$2\frac{11}{16}$	↓
×91.5^{c,v}	26.7	19.5	$19\frac{1}{2}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.7	11.8	$11\frac{3}{4}$	1.20	$1\frac{3}{16}$	2.38	$2\frac{1}{2}$	↓
×83.5^{c,v}	24.6	19.3	$19\frac{1}{4}$	0.650	$\frac{5}{8}$	$\frac{5}{16}$	12.5	11.8	$11\frac{3}{4}$	1.03	1	2.21	$2\frac{5}{16}$	↓
×74.5^{c,v}	21.9	19.1	$19\frac{1}{8}$	0.630	$\frac{5}{8}$	$\frac{5}{16}$	12.0	11.8	$11\frac{3}{4}$	0.830	$\frac{13}{16}$	2.01	$2\frac{1}{8}$	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.

[h] Flange thickness greater than 2 in. Special requirements may apply per Specification Section A3.1c.

[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1–8 (continued)
WT Shapes
Properties

WT22-WT20

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 50$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
167.5	4.50	21.5	2170	131	6.63	5.53	234	1.54	600	75.2	3.49	118	0.822	37.2	438
145	5.02	25.2	1830	111	6.54	5.26	196	1.35	521	65.9	3.49	102	0.629	25.4	275
131	5.57	27.6	1640	99.4	6.53	5.19	176	1.22	462	58.6	3.47	90.9	0.526	18.6	200
115	6.45	30.2	1440	88.6	6.53	5.17	157	1.07	398	50.5	3.43	78.3	0.438	12.4	139
296.5	2.58	12.0	3310	209	6.16	5.66	379	2.61	1260	151	3.80	240	1.00	221	2340
251.5	2.98	13.7	2730	174	6.07	5.38	314	2.25	1020	124	3.72	197	1.00	138	1400
215.5	3.44	15.4	2290	148	6.01	5.18	266	1.95	843	104	3.65	164	1.00	88.2	881
198.5	3.66	16.8	2070	134	5.96	5.03	240	1.81	771	95.7	3.63	150	1.00	70.6	677
186	3.93	17.5	1930	126	5.95	4.98	225	1.70	709	88.3	3.60	138	1.00	57.7	558
181	3.99	18.1	1870	122	5.92	4.91	217	1.66	691	86.3	3.60	135	0.993	54.2	511
162	4.40	20.1	1650	108	5.88	4.77	192	1.50	609	76.6	3.57	119	0.893	39.6	362
148.5	4.80	21.4	1500	98.9	5.87	4.71	176	1.38	546	69.0	3.54	107	0.825	30.5	279
138.5	5.03	23.9	1360	88.6	5.78	4.50	157	1.29	522	65.9	3.58	102	0.699	25.7	218
124.5	5.55	26.3	1210	79.4	5.75	4.41	140	1.16	463	58.8	3.55	90.8	0.580	19.0	158
107.5	6.45	30.0	1030	68.0	5.71	4.28	120	1.01	398	50.5	3.54	77.8	0.445	12.4	101
99.5	7.39	29.7	988	66.5	5.81	4.47	117	0.929	347	44.1	3.45	68.2	0.452	9.12	83.5
196	2.45	14.7	2270	153	6.27	5.94	275	2.33	401	64.9	2.64	106	1.00	85.4	796
165.5	2.86	16.7	1880	128	6.21	5.74	231	2.00	322	52.9	2.57	85.7	1.00	52.5	484
163.5	2.85	17.3	1840	125	6.19	5.66	224	1.98	320	52.7	2.58	85.0	1.00	51.4	449
147	3.11	19.1	1630	111	6.14	5.51	199	1.80	281	46.7	2.55	75.0	0.945	38.2	322
139	3.31	19.6	1550	106	6.14	5.51	191	1.71	261	43.5	2.52	69.9	0.918	32.4	282
132	3.45	20.8	1450	99.2	6.11	5.41	178	1.63	246	41.3	2.52	66.0	0.855	27.9	233
117.5	3.77	23.9	1260	85.7	6.04	5.17	153	1.45	222	37.3	2.54	59.0	0.699	20.6	156
105.5	4.17	26.2	1120	76.7	6.01	5.08	137	1.31	195	33.0	2.51	52.1	0.581	15.2	113
91.5	4.92	30.0	955	65.7	5.98	4.97	117	1.13	165	28.0	2.49	44.0	0.445	9.65	71.2
83.5	5.76	29.7	899	63.7	6.05	5.19	115	1.10	141	23.9	2.40	37.8	0.454	6.99	62.9
74.5	7.11	30.3	815	59.7	6.10	5.45	108	1.72	114	19.4	2.29	30.9	0.435	4.66	51.9

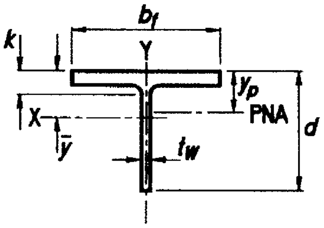


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT18×400^{h}	118	21.3	21 1/4	2.38	2 3/8	1 3/16	50.6	18.0	18	4.29	4 5/16	5.24	5 9/16	7 1/2
×326^{h}	96.1	20.5	20 1/2	1.97	2	1	40.4	17.6	17 5/8	3.54	3 9/16	4.49	4 13/16	
×264.5^{h}	77.8	19.9	19 7/8	1.61	1 5/8	13/16	32.0	17.2	17 1/4	2.91	2 15/16	3.86	4 3/16	
×243.5^{h}	71.7	19.7	19 5/8	1.50	1 1/2	3/4	29.5	17.1	17 1/8	2.68	2 11/16	3.63	4	
×220.5^{h}	64.9	19.4	19 3/8	1.36	1 3/8	11/16	26.4	17.0	17	2.44	2 7/16	3.39	3 3/4	
×197.5^{h}	58.2	19.2	19 1/4	1.22	1 1/4	5/8	23.4	16.8	16 7/8	2.20	2 3/16	3.15	3 7/16	
×180.5	53.0	19.0	19	1.12	1 1/8	9/16	21.3	16.7	16 3/4	2.01	2	2.96	3 5/16	
×165^{c}	48.5	18.8	18 7/8	1.02	1	1/2	19.2	16.6	16 5/8	1.85	1 7/8	2.80	3 1/8	
×151^{c}	44.4	18.7	18 5/8	0.945	15/16	1/2	17.6	16.7	16 5/8	1.68	1 11/16	2.63	3	
×141^{c}	41.5	18.6	18 1/2	0.885	7/8	7/16	16.4	16.6	16 5/8	1.57	1 9/16	2.52	2 7/8	
×131^{c}	38.5	18.4	18 3/8	0.840	13/16	7/16	15.5	16.6	16 1/2	1.44	1 7/16	2.39	2 3/4	
×123.5^{c}	36.3	18.3	18 3/8	0.800	13/16	7/16	14.7	16.5	16 1/2	1.35	1 3/8	2.30	2 5/8	
×115.5^{c}	34.0	18.2	18 1/4	0.760	3/4	3/8	13.9	16.5	16 1/2	1.26	1 1/4	2.21	2 9/16	
WT18×128^{c}	37.7	18.7	18 3/4	0.960	15/16	1/2	18.0	12.2	12 1/4	1.73	1 3/4	2.48	2 5/8	5 1/2
×116^{c}	34.1	18.6	18 1/2	0.870	7/8	7/16	16.1	12.1	12 1/8	1.57	1 9/16	2.32	2 7/16	
×105^{c}	30.9	18.3	18 3/8	0.830	13/16	7/16	15.2	12.2	12 1/8	1.36	1 3/8	2.11	2 5/16	
×97^{c}	28.5	18.2	18 1/4	0.765	3/4	3/8	14.0	12.1	12 1/8	1.26	1 1/4	2.01	2 3/16	
×91^{c}	26.8	18.2	18 1/8	0.725	3/4	3/8	13.2	12.1	12 1/8	1.18	1 3/16	1.93	2 1/8	
×85^{c}	25.0	18.1	18 1/8	0.680	11/16	3/8	12.3	12.0	12	1.10	1 1/8	1.85	2	
×80^{c}	23.5	18.0	18	0.650	5/8	5/16	11.7	12.0	12	1.02	1	1.77	1 15/16	
×75^{c}	22.1	17.9	17 7/8	0.625	5/8	5/16	11.2	12.0	12	0.940	15/16	1.69	1 7/8	
×67.5^{c,v}	19.9	17.8	17 3/4	0.600	5/8	5/16	10.7	12.0	12	0.790	13/16	1.54	1 11/16	
WT16.5×193.5^{h}	57.0	18.0	18	1.26	1 1/4	5/8	22.6	16.2	16 1/4	2.28	2 1/4	3.07	3 3/16	5 1/2
×177^{h}	52.1	17.8	17 3/4	1.16	1 3/16	5/8	20.6	16.1	16 1/8	2.09	2 1/16	2.88	2 15/16	
×159	46.8	17.6	17 5/8	1.04	1 1/16	9/16	18.3	16.0	16	1.89	1 7/8	2.68	2 3/4	
×145.5^{c}	42.8	17.4	17 3/8	0.960	15/16	1/2	16.7	15.9	15 7/8	1.73	1 3/4	2.52	2 5/8	
×131.5^{c}	38.7	17.3	17 1/4	0.870	7/8	7/16	15.0	15.8	15 3/4	1.57	1 9/16	2.36	2 7/16	
×120.5^{c}	35.5	17.1	17 1/8	0.830	13/16	7/16	14.2	15.9	15 7/8	1.40	1 3/8	2.19	2 1/4	
×110.5^{c}	32.6	17.0	17	0.775	3/4	3/8	13.1	15.8	15 3/4	1.28	1 1/4	2.06	2 1/8	
×100.5^{c}	29.6	16.8	16 7/8	0.715	11/16	3/8	12.0	15.7	15 3/4	1.15	1 1/8	1.94	2	

[c] Shape is slender for compression with $F_y = 50$ ksi.
[h] Flange thickness greater than 2 in. Special requirements may apply per Specification Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1–8 (continued)
WT Shapes
Properties

WT18-WT16.5

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	F_y = 50 ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
400	2.10	8.94	4090	264	5.89	5.80	491	3.28	2100	234	4.22	371	1.00	525	5810
326	2.48	10.4	3160	208	5.74	5.35	383	2.73	1610	184	4.10	290	1.00	295	3070
264.5	2.96	12.4	2440	164	5.60	4.96	298	2.26	1240	145	4.00	227	1.00	163	1600
243.5	3.19	13.1	2220	150	5.57	4.84	272	2.10	1120	131	3.96	206	1.00	128	1250
220.5	3.48	14.3	1980	134	5.52	4.69	242	1.91	997	117	3.92	184	1.00	96.6	914
197.5	3.83	15.7	1740	119	5.47	4.53	213	1.73	877	104	3.88	162	1.00	70.7	652
180.5	4.16	17.0	1570	107	5.43	4.42	192	1.59	786	94.0	3.85	146	1.00	54.1	491
165	4.49	18.5	1410	97.0	5.39	4.30	173	1.46	711	85.5	3.83	132	0.974	42.0	372
151	4.96	19.8	1280	88.8	5.37	4.22	158	1.33	648	77.8	3.82	120	0.909	32.1	285
141	5.29	21.0	1190	82.6	5.36	4.16	146	1.25	599	72.2	3.80	112	0.848	26.3	231
131	5.75	21.9	1110	77.5	5.36	4.14	137	1.16	545	65.8	3.76	102	0.799	20.8	185
123.5	6.11	22.9	1040	73.3	5.36	4.12	129	1.10	507	61.4	3.74	94.8	0.749	17.3	155
116	6.54	24.0	978	69.1	5.36	4.10	122	1.03	470	57.0	3.71	88.0	0.694	14.3	129
128	3.53	19.5	1210	87.4	5.66	4.92	156	1.54	264	43.2	2.65	68.5	0.922	26.4	205
116	3.86	21.3	1080	78.5	5.63	4.82	140	1.40	234	38.6	2.62	60.9	0.829	19.7	151
105	4.48	22.1	985	73.1	5.65	4.87	131	1.27	206	33.8	2.58	53.4	0.791	13.9	119
97	4.81	23.8	901	67.0	5.62	4.80	120	1.18	187	30.9	2.56	48.8	0.702	11.1	92.7
91	5.12	25.1	845	63.1	5.62	4.77	113	1.11	174	28.8	2.55	45.3	0.637	9.20	77.6
85	5.47	26.6	786	58.9	5.61	4.73	105	1.04	160	26.6	2.53	41.8	0.566	7.51	63.2
80	5.88	27.7	740	55.8	5.61	4.74	100	0.980	147	24.6	2.50	38.6	0.521	6.17	53.6
75	6.37	28.7	698	53.1	5.62	4.78	95.5	0.923	135	22.5	2.47	35.4	0.486	5.04	46.0
67.5	7.56	29.6	637	49.7	5.66	4.96	90.1	1.23	113	18.9	2.38	29.8	0.456	3.48	37.3
193.5	3.55	14.3	1460	107	5.07	4.27	193	1.76	810	100	3.77	156	1.00	73.9	615
177	3.85	15.3	1320	96.8	5.03	4.15	174	1.62	729	90.6	3.74	141	1.00	57.1	468
159	4.23	16.9	1160	85.8	4.99	4.02	154	1.46	645	80.7	3.71	125	1.00	42.1	335
145.5	4.60	18.1	1060	78.3	4.96	3.93	140	1.35	581	73.1	3.68	113	0.991	32.5	256
131.5	5.03	19.8	943	70.2	4.93	3.83	125	1.23	517	65.5	3.65	101	0.905	24.3	188
120.5	5.66	20.6	872	65.8	4.96	3.84	116	1.12	466	58.8	3.62	90.8	0.867	18.0	146
110.5	6.20	21.9	799	60.8	4.95	3.81	107	1.03	420	53.2	3.59	82.1	0.801	13.9	113
100.5	6.85	23.6	725	55.5	4.95	3.77	97.8	0.940	375	47.6	3.56	73.3	0.717	10.4	84.9

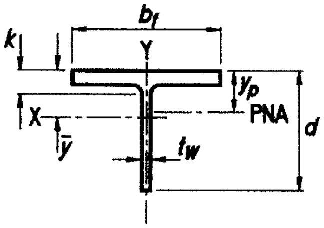


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT16.5×84.5^{c}	24.8	16.9	16 7/8	0.670	11/16	3/8	11.3	11.5	11 1/2	1.22	1 1/4	1.92	2 1/8	5 1/2
×76^{c}	22.4	16.7	16 3/4	0.635	5/8	5/16	10.6	11.6	11 5/8	1.06	1 1/16	1.76	1 15/16	
×70.5^{c}	20.8	16.7	16 5/8	0.605	5/8	5/16	10.1	11.5	11 1/2	0.960	15/16	1.66	1 13/16	
×65^{c}	19.2	16.5	16 1/2	0.580	9/16	5/16	9.60	11.5	11 1/2	0.855	7/8	1.56	1 3/4	
×59^{c,v}	17.3	16.4	16 3/8	0.550	9/16	5/16	9.04	11.5	11 1/2	0.740	3/4	1.44	5/8	↓
WT15×195.5^{h}	57.6	16.6	16 5/8	1.36	1 3/8	11/16	22.6	15.6	15 5/8	2.44	2 7/16	3.23	3 3/8	5 1/2
×178.5^{h}	52.5	16.4	16 3/8	1.24	1 1/4	5/8	20.3	15.5	15 1/2	2.24	2 1/4	3.03	3 1/8	
×163^{h}	47.9	16.2	16 1/4	1.14	1 1/8	9/16	18.5	15.4	15 3/8	2.05	2 1/16	2.84	2 15/16	
×146	42.9	16.0	16	1.02	1	1/2	16.3	15.3	15 1/4	1.85	1 7/8	2.64	2 3/4	
×130.5	38.4	15.8	15 3/4	0.930	15/16	1/2	14.7	15.2	15 1/8	1.65	1 5/8	2.44	2 9/16	
×117.5^{c}	34.6	15.7	15 5/8	0.830	13/16	7/16	13.0	15.1	15	1.50	1 1/2	2.29	2 3/8	
×105.5^{c}	31.1	15.5	15 1/2	0.775	3/4	3/8	12.0	15.1	15 1/8	1.32	1 5/16	2.10	2 1/4	
×95.5^{c}	28.1	15.3	15 3/8	0.710	11/16	3/8	10.9	15.0	15	1.19	1 3/16	1.97	2 1/16	↓
WT15×86.5^{c}	25.5	15.2	15 1/4	0.655	5/8	5/16	10.0	15.0	15	1.07	1 1/16	1.85	2	5 1/2
×74^{c}	21.7	15.3	15 3/8	0.650	5/8	5/16	10.0	10.5	10 1/2	1.18	1 3/16	1.83	2 1/16	
×66^{c}	19.4	15.2	15 1/8	0.615	5/8	5/16	9.32	10.5	10 1/2	1.00	1	1.65	1 7/8	
×62^{c}	18.2	15.1	15 1/8	0.585	9/16	5/16	8.82	10.5	10 1/2	0.930	15/16	1.58	1 13/16	
×58^{c}	17.1	15.0	15	0.565	9/16	5/16	8.48	10.5	10 1/2	0.850	7/8	1.50	1 3/4	
×54^{c}	15.9	14.9	14 7/8	0.545	9/16	5/16	8.13	10.5	10 1/2	0.760	3/4	1.41	1 11/16	
×49.5^{c}	14.5	14.8	14 7/8	0.520	1/2	1/4	7.71	10.5	10 1/2	0.670	11/16	1.32	1 9/16	
×45^{c,v}	13.2	14.8	14 3/4	0.470	1/2	1/4	6.94	10.4	10 3/8	0.610	5/8	1.26	1 1/2	↓
WT13.5×269.5^{h}	79.3	16.3	16 1/4	1.97	2	1	32.0	15.3	15 1/4	3.54	3 9/16	4.33	4 7/16	5 1/2^{g}
×184^{h}	54.2	15.2	15 1/4	1.38	1 3/8	11/16	21.0	14.7	14 5/8	2.48	2 1/2	3.27	3 3/8	5 1/2
×168^{h}	49.5	15.0	15	1.26	1 1/4	5/8	18.9	14.6	14 1/2	2.28	2 1/4	3.07	3 3/16	
×153.5^{h}	45.2	14.8	14 3/4	1.16	1 3/16	5/8	17.2	14.4	14 1/2	2.09	2 1/16	2.88	3	
×140.5	41.4	14.6	14 5/8	1.06	1 1/16	9/16	15.5	14.4	14 3/8	1.93	1 15/16	2.72	2 13/16	
×129	38.0	14.5	14 1/2	0.980	1	1/2	14.2	14.3	14 1/4	1.77	1 3/4	2.56	2 11/16	
×117.5	34.7	14.3	14 3/8	0.910	15/16	1/2	13.0	14.2	14 1/4	1.61	1 5/8	2.40	2 1/2	
×108.5	32.0	14.2	14 1/4	0.830	13/16	7/16	11.8	14.1	14 1/8	1.50	1 1/2	2.29	2 3/8	
×97^{c}	28.6	14.1	14	0.750	3/4	3/8	10.5	14.0	14	1.34	1 5/16	2.13	2 1/4	
×89^{c}	26.2	13.9	13 7/8	0.725	3/4	3/8	10.1	14.1	14 1/8	1.19	1 3/16	1.98	2 1/16	
×80.5^{c}	23.8	13.8	13 3/4	0.660	11/16	3/8	9.10	14.0	14	1.08	1 1/16	1.87	2	
×73^{c}	21.6	13.7	13 3/4	0.605	5/8	5/16	8.28	14.0	14	0.975	1	1.76	1 7/8	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per Specification Section A3.1c.

[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT Shapes
Properties

WT16.5-WT13.5

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 50$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
84.5	4.71	25.2	649	51.1	5.12	4.21	90.8	1.08	155	27.0	2.50	42.1	0.628	8.81	55.4
76	5.48	26.4	592	47.4	5.14	4.26	84.5	0.967	136	23.6	2.47	36.9	0.575	6.16	43.0
70.5	6.01	27.5	552	44.7	5.15	4.29	79.8	0.901	123	21.3	2.43	33.4	0.528	4.84	35.4
65	6.73	28.5	513	42.1	5.18	4.36	75.6	0.832	109	18.9	2.38	29.7	0.492	3.67	29.3
59	7.76	29.9	469	39.2	5.20	4.47	70.8	0.862	93.5	16.3	2.32	25.6	0.448	2.64	23.4
195.5	3.19	12.2	1220	96.9	4.61	4.00	177	1.85	774	99.2	3.67	155	1.00	86.3	636
178.5	3.45	13.2	1090	87.2	4.56	3.87	159	1.70	693	89.6	3.64	140	1.00	66.6	478
163	3.75	14.2	981	78.8	4.52	3.76	143	1.56	622	81.0	3.60	126	1.00	51.2	361
146	4.12	15.7	861	69.6	4.48	3.62	125	1.41	549	71.9	3.58	111	1.00	37.5	257
130.5	4.59	17.0	765	62.4	4.46	3.54	112	1.27	480	63.3	3.53	97.9	1.00	26.9	184
117.5	5.02	18.9	674	55.1	4.41	3.41	98.2	1.15	427	56.8	3.51	87.5	0.955	20.1	133
105.5	5.74	20.0	610	50.5	4.43	3.39	89.5	1.03	378	50.1	3.49	77.2	0.899	14.1	96.4
95.5	6.35	21.6	549	45.7	4.42	3.34	80.8	0.935	336	44.7	3.46	68.9	0.816	10.5	71.2
86.5	7.04	23.2	497	41.7	4.42	3.31	73.5	0.851	299	39.9	3.42	61.4	0.733	7.78	53.0
74	4.44	23.6	466	40.6	4.63	3.84	72.2	1.04	114	21.7	2.28	33.9	0.715	7.24	37.6
66	5.27	24.6	421	37.4	4.66	3.90	66.8	0.921	98.0	18.6	2.25	29.2	0.662	4.85	28.5
62	5.65	25.8	396	35.3	4.66	3.90	63.1	0.867	90.4	17.2	2.23	27.0	0.602	3.98	23.9
58	6.17	26.6	373	33.7	4.67	3.94	60.4	0.815	82.1	15.6	2.19	24.6	0.567	3.21	20.5
54	6.89	27.4	349	32.0	4.69	4.01	57.7	0.757	73.0	13.9	2.15	21.9	0.534	2.49	17.3
49.5	7.80	28.5	322	30.0	4.71	4.09	54.4	0.912	63.9	12.2	2.10	19.3	0.492	1.88	14.3
45	8.52	31.4	290	27.1	4.69	4.04	49.0	0.835	57.3	11.0	2.09	17.3	0.405	1.41	10.5
269.5	2.15	8.25	1530	128	4.39	4.34	242	2.60	1060	138	3.65	218	1.00	247	1740
184	2.96	11.0	939	81.7	4.16	3.71	151	1.85	655	89.3	3.48	140	1.00	84.5	532
168	3.19	11.9	839	73.4	4.12	3.58	135	1.70	587	80.8	3.45	126	1.00	65.4	401
153.5	3.46	12.8	753	66.4	4.08	3.47	121	1.56	527	72.9	3.41	113	1.00	50.5	304
140.5	3.72	13.8	677	59.9	4.04	3.35	109	1.44	477	66.4	3.39	103	1.00	39.6	232
129	4.03	14.8	613	54.7	4.02	3.27	98.9	1.33	430	60.2	3.36	93.3	1.00	30.7	178
117.5	4.41	15.7	556	50.0	4.00	3.20	89.9	1.22	384	54.2	3.33	83.8	1.00	23.4	135
108.5	4.71	17.1	502	45.2	3.96	3.10	81.1	1.13	352	49.9	3.32	77.0	1.00	18.8	105
97	5.24	18.7	444	40.3	3.94	3.02	71.8	1.02	309	44.1	3.29	67.8	0.961	13.5	74.3
89	5.92	19.2	414	38.2	3.97	3.04	67.7	0.932	278	39.4	3.25	60.8	0.938	10.0	57.7
80.5	6.49	20.9	372	34.4	3.95	2.98	60.8	0.849	248	35.4	3.23	54.5	0.851	7.53	42.7
73	7.16	22.6	336	31.2	3.95	2.94	55.0	0.772	222	31.7	3.20	48.8	0.764	5.62	31.7

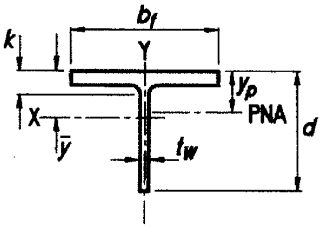


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem: Thickness, t_w		Stem: $\frac{t_w}{2}$	Stem: Area	Flange: Width, b_f		Flange: Thickness, t_f		Distance: k		Distance: Workable Gage
	in.²	in.		in.		in.	in.²	in.		in.		in.	in.	in.
WT13.5×64.5[c]	18.9	13.8	13 7/8	0.610	5/8	5/16	8.43	10.0	10	1.10	1 1/8	1.70	2	5 1/2
×57[c]	16.8	13.6	13 5/8	0.570	9/16	5/16	7.78	10.1	10 1/8	0.930	15/16	1.53	1 13/16	↓
×51[c]	15.0	13.5	13 1/2	0.515	1/2	1/4	6.98	10.0	10	0.830	13/16	1.43	1 3/4	↓
×47[c]	13.8	13.5	13 1/2	0.490	1/2	1/4	6.60	10.0	10	0.745	3/4	1.34	1 5/8	↓
×42[c]	12.4	13.4	13 3/8	0.460	7/16	1/4	6.14	10.0	10	0.640	5/8	1.24	1 9/16	↓
WT12×185[h]	54.4	14.0	14	1.52	1 1/2	3/4	21.3	13.7	13 5/8	2.72	2 3/4	3.22	3 5/8	5 1/2
×167.5[h]	49.2	13.8	13 3/4	1.38	1 3/8	11/16	19.0	13.5	13 1/2	2.48	2 1/2	2.98	3 3/8	↓
×153[h]	44.9	13.6	13 5/8	1.26	1 1/4	5/8	17.1	13.4	13 3/8	2.28	2 1/4	2.78	3 3/16	↓
×139.5[h]	41.0	13.4	13 3/8	1.16	1 3/16	5/8	15.5	13.3	13 1/4	2.09	2 1/16	2.59	3	↓
×125	36.8	13.2	13 1/8	1.04	1 1/16	9/16	13.7	13.2	13 1/8	1.89	1 7/8	2.39	2 13/16	↓
×114.5	33.6	13.0	13	0.960	15/16	1/2	12.5	13.1	13 1/8	1.73	1 3/4	2.23	2 5/8	↓
×103.5	30.4	12.9	12 7/8	0.870	7/8	7/16	11.2	13.0	13	1.57	1 9/16	2.07	2 1/2	↓
×96	28.1	12.7	12 3/4	0.810	13/16	7/16	10.3	13.0	13	1.46	1 7/16	1.96	2 3/8	↓
×88	25.8	12.6	12 5/8	0.750	3/4	3/8	9.47	12.9	12 7/8	1.34	1 5/16	1.84	2 1/4	↓
×81	23.9	12.5	12 1/2	0.705	11/16	3/8	8.81	13.0	13	1.22	1 1/4	1.72	2 1/8	↓
×73[c]	21.5	12.4	12 3/8	0.650	5/8	5/16	8.04	12.9	12 7/8	1.09	1 1/16	1.59	2	↓
×65.5[c]	19.3	12.2	12 1/4	0.605	5/8	5/16	7.41	12.9	12 /8	0.960	15/16	1.46	1 7/8	↓
×58.5[c]	17.2	12.1	12 1/8	0.550	9/16	5/16	6.67	12.8	12 3/4	0.850	7/8	1.35	1 3/4	↓
×52[c]	15.3	12.0	12	0.500	1/2	1/4	6.02	12.8	12 3/4	0.750	3/4	1.25	1 5/8	↓
WT12×51.5[c]	15.1	12.3	12 1/4	0.550	9/16	5/16	6.75	9.00	9	0.980	1	1.48	1 7/8	5 1/2
×47[c]	13.8	12.2	12 1/8	0.515	1/2	1/4	6.26	9.07	9 1/8	0.875	7/8	1.38	1 3/4	↓
×42[c]	12.4	12.1	12	0.470	1/2	1/4	5.66	9.02	9	0.770	3/4	1.27	1 11/16	↓
×38[c]	11.2	12.0	12	0.440	7/16	1/4	5.26	8.99	9	0.680	11/16	1.18	1 9/16	5 1/2[g]
×34[c]	10.0	11.9	11 7/8	0.415	7/16	1/4	4.92	8.97	9	0.585	9/16	1.09	1 1/2	5 1/2[g]
WT12×31[c]	9.11	11.9	11 7/8	0.430	7/16	1/4	5.10	7.04	7	0.590	9/16	1.09	1 1/2	3 1/2
×27.5[c,v]	8.10	11.8	11 3/4	0.395	3/8	3/16	4.66	7.01	7	0.505	1/2	1.01	1 7/16	3 1/2

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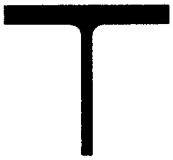
[c] Shape is slender for compression with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per Specification Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1-8 (continued)
WT Shapes
Properties



WT13.5-WT12

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 50$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
64.5	4.55	22.6	323	31.0	4.13	3.39	55.1	0.945	92.2	18.4	2.21	28.8	0.763	5.55	24.0
57	5.41	23.9	289	28.3	4.15	3.42	50.4	0.832	79.3	15.8	2.18	24.6	0.698	3.65	17.5
51	6.03	26.3	258	25.3	4.14	3.37	45.0	0.750	69.6	13.9	2.15	21.7	0.578	2.63	12.6
47	6.70	27.5	239	23.8	4.16	3.41	42.4	0.692	62.0	12.4	2.12	19.4	0.530	2.01	10.2
42	7.78	29.0	216	21.9	4.18	3.48	39.2	0.621	52.8	10.6	2.07	16.6	0.475	1.40	7.79
185	2.51	9.21	779	74.7	3.78	3.57	140	1.99	581	85.1	3.27	133	1.00	100	553
167.5	2.73	10.0	686	66.3	3.73	3.42	123	1.82	513	75.9	3.23	119	1.00	75.6	405
153	2.94	10.8	611	59.4	3.69	3.29	110	1.67	460	68.6	3.20	107	1.00	58.4	305
139.5	3.18	11.5	546	53.6	3.65	3.18	98.8	1.54	412	61.9	3.17	96.3	1.00	45.1	230
125	3.49	12.7	478	47.2	3.61	3.05	86.5	1.39	362	54.9	3.14	85.2	1.00	33.2	165
114.5	3.79	13.6	431	42.9	3.58	2.96	78.1	1.28	326	49.7	3.11	77.0	1.00	25.5	125
103.5	4.14	14.8	382	38.3	3.55	2.87	69.3	1.17	289	44.4	3.08	68.6	1.00	19.1	91.3
96	4.43	15.7	350	35.2	3.53	2.80	63.5	1.09	265	40.9	3.07	63.1	1.00	15.3	72.5
88	4.81	16.8	319	32.2	3.51	2.74	57.8	1.00	240	37.2	3.04	57.3	1.00	11.9	55.8
81	5.31	17.7	293	29.9	3.50	2.70	53.3	0.921	221	34.2	3.05	52.6	1.00	9.22	43.8
73	5.92	19.0	264	27.2	3.50	2.66	48.2	0.833	195	30.3	3.01	46.6	0.946	6.70	31.9
65.5	6.70	20.2	238	24.8	3.52	2.65	43.9	0.750	170	26.5	2.97	40.7	0.885	4.74	23.1
58.5	7.53	22.1	212	22.3	3.51	2.62	39.2	0.672	149	23.2	2.94	35.7	0.793	3.35	16.4
52	8.50	24.1	189	20.0	3.51	2.59	35.1	0.600	130	20.3	2.91	31.2	0.692	2.35	11.6
51.5	4.59	22.3	204	22.0	3.67	3.01	39.2	0.841	59.7	13.3	1.99	20.7	0.781	3.53	12.3
47	5.18	23.6	186	20.3	3.67	2.99	36.1	0.764	54.5	12.0	1.98	18.7	0.715	2.62	9.57
42	5.86	25.6	166	18.3	3.67	2.97	32.5	0.685	47.2	10.5	1.95	16.3	0.609	1.84	6.90
38	6.61	27.2	151	16.9	3.68	3.00	30.1	0.622	41.3	9.18	1.92	14.3	0.541	1.34	5.30
34	7.66	28.6	137	15.6	3.70	3.06	27.9	0.560	35.2	7.85	1.87	12.3	0.489	0.932	4.08
31	5.97	27.6	131	15.6	3.79	3.46	28.4	1.28	17.2	4.90	1.38	7.85	0.525	0.850	3.92
27.5	6.94	29.8	117	14.1	3.80	3.50	25.6	1.53	14.5	4.15	1.34	6.65	0.449	0.588	2.93

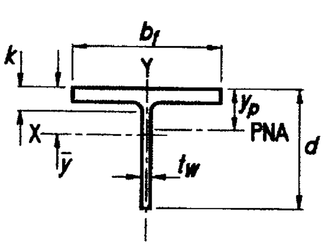


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT10.5×100.5	29.6	11.5	11 1/2	0.910	15/16	1/2	10.5	12.6	12 5/8	1.63	1 5/8	2.13	2 1/2	5 1/2
×91	26.8	11.4	11 3/8	0.830	13/16	7/16	9.43	12.5	12 1/2	1.48	1 1/2	1.98	2 3/8	↓
×83	24.4	11.2	11 1/4	0.750	3/4	3/8	8.43	12.4	12 3/8	1.36	1 3/8	1.86	2 1/4	
×73.5	21.6	11.0	11	0.720	3/4	3/8	7.94	12.5	12 1/2	1.15	1 1/8	1.65	2	
×66	19.4	10.9	10 7/8	0.650	5/8	5/16	7.09	12.4	12 1/2	1.04	1 1/16	1.54	1 15/16	
×61[c]	17.9	10.8	10 7/8	0.600	5/8	5/16	6.50	12.4	12 3/8	0.960	15/16	1.46	1 13/16	
×55.5[c]	16.3	10.8	10 3/4	0.550	9/16	5/16	5.92	12.3	12 3/8	0.875	7/8	1.38	1 3/4	
×50.5[c]	14.9	10.7	10 5/8	0.500	1/2	1/4	5.34	12.3	12 1/4	0.800	13/16	1.30	1 11/16	
WT10.5×46.5[c]	13.7	10.8	10 3/4	0.580	9/16	5/16	6.27	8.42	8 3/8	0.930	15/16	1.43	1 5/8	5 1/2
×41.5[c]	12.2	10.7	10 3/4	0.515	1/2	1/4	5.52	8.36	8 3/8	0.835	13/16	1.34	1 1/2	↓
×36.5[c]	10.7	10.6	10 5/8	0.455	7/16	1/4	4.83	8.30	8 1/4	0.740	3/4	1.24	1 7/16	
×34[c]	10.0	10.6	10 5/8	0.430	7/16	1/4	4.54	8.27	8 1/4	0.685	11/16	1.19	1 3/8	
×31[c]	9.13	10.5	10 1/2	0.400	3/8	3/16	4.20	8.24	8 1/4	0.615	5/8	1.12	1 5/16	
×27.5[c]	8.10	10.4	10 3/8	0.375	3/8	3/16	3.90	8.22	8 1/4	0.522	1/2	1.02	1 3/16	
×24[c,f,v]	7.07	10.3	10 1/4	0.350	3/8	3/16	3.61	8.14	8 1/8	0.430	7/16	0.930	1 1/8	
WT10.5×28.5[c,h]	8.37	10.5	10 1/2	0.405	3/8	3/16	4.26	6.56	6 1/2	0.650	5/8	1.15	1 5/16	3 1/2
×25[c]	7.36	10.4	10 3/8	0.380	3/8	3/16	3.96	6.53	6 1/2	0.535	9/16	1.04	1 1/4	3 1/2[g]
×22[c,v]	6.49	10.3	10 3/8	0.350	3/8	3/16	3.62	6.50	6 1/2	0.450	7/16	0.950	1 1/8	3 1/2[g]
WT9×155.5[h]	45.8	11.2	11 1/8	1.52	1 1/2	3/4	17.0	12.0	12	2.74	2 3/4	3.24	3 7/16	5 1/2
×141.5[h]	41.6	10.9	10 7/8	1.40	1 3/8	11/16	15.3	11.9	11 7/8	2.50	2 1/2	3.00	3 3/16	↓
×129[h]	37.9	10.7	10 3/4	1.28	1 1/4	5/8	13.7	11.8	11 3/4	2.30	2 5/16	2.70	3	
×117[h]	34.4	10.5	10 1/2	1.16	1 3/16	5/8	12.2	11.7	11 5/8	2.11	2 1/8	2.51	2 3/4	
×105.5	31.1	10.3	10 3/8	1.06	1 1/16	9/16	11.0	11.6	11 1/2	1.91	1 15/16	2.31	2 9/16	
×96	28.2	10.2	10 1/8	0.960	15/16	1/2	9.77	11.5	11 1/2	1.75	1 3/4	2.15	2 7/16	
×87.5	25.7	10.0	10	0.890	7/8	7/16	8.92	11.4	11 3/8	1.59	1 9/16	1.99	2 7/16	
×79	23.2	9.86	9 7/8	0.810	13/16	7/16	7.99	11.3	11 1/4	1.44	1 7/16	1.84	2 3/8	
×71.5	21.0	9.75	9 3/4	0.730	3/4	3/8	7.11	11.2	11 1/4	1.32	1 5/16	1.72	2 3/16	
×65	19.1	9.63	9 5/8	0.670	11/16	3/8	6.45	11.2	11 1/8	1.20	1 3/16	1.60	2 1/16	
×59.5	17.5	9.49	9 1/2	0.655	5/8	5/16	6.21	11.3	11 1/4	1.06	1 1/16	1.46	1 15/16	
×53	15.6	9.37	9 3/8	0.590	9/16	5/16	5.53	11.2	11 1/4	0.940	15/16	1.34	1 13/16	
×48.5	14.3	9.30	9 1/4	0.535	9/16	5/16	4.97	11.1	11 1/8	0.870	7/8	1.27	1 3/4	
×43[c]	12.7	9.20	9 1/4	0.480	1/2	1/4	4.41	11.1	11 1/8	0.770	3/4	1.17	1 5/8	
×38[c]	11.2	9.11	9 1/8	0.425	7/16	1/4	3.87	11.0	11	0.680	11/16	1.08	1 9/16	

[c] Shape is slender for compression with $F_y = 50$ ksi.

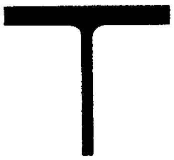
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per Specification Section A3.1c.

[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1–8 (continued)
WT Shapes
Properties



WT10.5-WT9

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 50$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
100.5	3.86	12.7	285	31.9	3.10	2.57	58.6	1.18	271	43.1	3.02	66.5	1.00	20.4	85.4
91	4.22	13.7	253	28.5	3.07	2.48	52.1	1.07	241	38.6	3.00	59.5	1.00	15.3	63.0
83	4.57	15.0	226	25.5	3.04	2.39	46.3	0.983	217	35.0	2.99	53.9	1.00	11.8	47.3
73.5	5.44	15.3	204	23.7	3.08	2.39	42.4	0.864	188	30.0	2.95	46.3	1.00	7.69	32.5
66	6.01	16.8	181	21.1	3.06	2.33	37.6	0.780	166	26.7	2.93	41.1	1.00	5.62	23.4
61	6.45	18.1	166	19.3	3.04	2.28	34.3	0.724	152	24.6	2.91	37.8	0.995	4.47	18.4
55.5	7.05	19.6	150	17.5	3.03	2.23	31.0	0.662	137	22.2	2.90	34.1	0.919	3.40	13.8
50.5	7.68	21.4	135	15.8	3.01	2.18	27.9	0.605	124	20.2	2.89	30.8	0.828	2.60	10.4
46.5	4.53	18.6	144	17.9	3.25	2.74	31.8	0.812	46.4	11.0	1.84	17.3	0.966	3.01	9.33
41.5	5.00	20.8	127	15.7	3.22	2.66	28.0	0.728	40.7	9.74	1.83	15.2	0.856	2.16	6.50
36.5	5.60	23.3	110	13.8	3.21	2.60	24.4	0.647	35.3	8.51	1.81	13.3	0.728	1.51	4.42
34	6.04	24.6	103	12.9	3.20	2.59	22.9	0.606	32.4	7.83	1.80	12.2	0.666	1.22	3.62
31	6.70	26.2	93.8	11.9	3.21	2.58	21.1	0.554	28.7	6.97	1.77	10.9	0.581	0.913	2.78
27.5	7.87	27.7	84.4	10.9	3.23	2.64	19.4	0.493	24.2	5.89	1.73	9.18	0.520	0.617	2.08
24	9.47	29.5	74.9	9.90	3.26	2.74	17.8	0.459	19.4	4.76	1.66	7.44	0.461	0.400	1.52
28.5	5.04	26.0	90.4	11.8	3.29	2.85	21.2	0.638	15.3	4.67	1.35	7.40	0.592	0.884	2.50
25	6.10	27.4	80.3	10.7	3.30	2.93	19.4	0.771	12.5	3.82	1.30	6.08	0.532	0.570	1.89
22	7.22	29.5	71.1	9.68	3.31	2.98	17.6	1.06	10.3	3.18	1.26	5.07	0.459	0.383	1.40
155.5	2.19	7.34	383	46.6	2.89	2.93	90.6	1.91	398	66.2	2.95	104	1.00	87.2	339
141.5	2.38	7.80	337	41.5	2.85	2.80	80.2	1.75	352	59.2	2.91	92.5	1.00	66.5	251
129	2.56	8.38	298	37.0	2.80	2.68	71.0	1.61	314	53.4	2.88	83.1	1.00	51.1	189
117	2.76	9.08	261	32.7	2.75	2.55	62.4	1.48	279	47.9	2.85	74.4	1.00	39.1	140
105.5	3.02	9.75	229	29.1	2.72	2.44	55.0	1.34	246	42.7	2.82	66.1	1.00	29.1	102
96	3.27	10.6	202	25.8	2.68	2.34	48.5	1.23	220	38.4	2.79	59.4	1.00	22.3	75.7
87.5	3.58	11.3	181	23.4	2.66	2.26	43.6	1.13	196	34.4	2.76	53.1	1.00	16.8	56.5
79	3.92	12.2	160	20.8	2.63	2.17	38.5	1.02	174	30.7	2.74	47.4	1.00	12.5	41.2
71.5	4.25	13.3	142	18.5	2.60	2.09	34.0	0.937	156	27.7	2.72	42.7	1.00	9.58	30.7
65	4.65	14.4	127	16.7	2.58	2.02	30.5	0.856	139	24.9	2.70	38.3	1.00	7.23	22.8
59.5	5.31	14.5	119	15.9	2.60	2.03	28.7	0.778	126	22.5	2.69	34.5	1.00	5.30	17.4
53	5.96	15.9	104	14.1	2.59	1.97	25.2	0.695	110	19.7	2.66	30.2	1.00	3.73	12.1
48.5	6.41	17.4	93.8	12.7	2.56	1.91	22.6	0.640	100	18.0	2.65	27.6	1.00	2.92	9.29
43	7.20	19.2	82.4	11.2	2.55	1.86	19.9	0.570	87.6	15.8	2.63	24.2	0.939	2.04	6.42
38	8.11	21.4	71.8	9.83	2.54	1.80	17.3	0.505	76.2	13.8	2.61	21.1	0.825	1.41	4.37

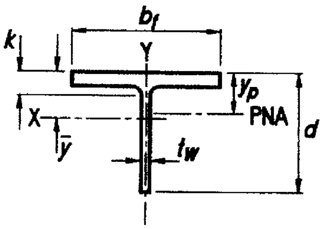


Table 1-8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT9×35.5[c]	10.4	9.24	9 1/4	0.495	1/2	1/4	4.57	7.64	7 5/8	0.810	13/16	1.21	1 1/2	3 1/2[g]
×32.5[c]	9.55	9.18	9 1/8	0.450	7/16	1/4	4.13	7.59	7 5/8	0.750	3/4	1.15	1 7/16	↓
×30[c]	8.82	9.12	9 1/8	0.415	7/16	1/4	3.78	7.56	7 1/2	0.695	11/16	1.10	1 3/8	
×27.5[c]	8.10	9.06	9	0.390	3/8	3/16	3.53	7.53	7 1/2	0.630	5/8	1.03	1 5/16	
×25[c]	7.33	9.00	9	0.355	3/8	3/16	3.19	7.50	7 1/2	0.570	9/16	0.972	1 1/4	
WT9×23[c]	6.77	9.03	9	0.360	3/8	3/16	3.25	6.06	6	0.605	5/8	1.01	1 1/4	3 1/2[g]
×20[c]	5.88	8.95	9	0.315	5/16	3/16	2.82	6.02	6	0.525	1/2	0.927	1 3/16	↓
×17.5[c,v]	5.15	8.85	8 7/8	0.300	5/16	3/16	2.66	6.00	6	0.425	7/16	0.827	1 1/8	
WT8×50	14.7	8.49	8 1/2	0.585	9/16	5/16	4.96	10.4	10 3/8	0.985	1	1.39	1 7/8	5 1/2
×44.5	13.1	8.38	8 3/8	0.525	1/2	1/4	4.40	10.4	10 3/8	0.875	7/8	1.28	1 3/4	↓
×38.5[c]	11.3	8.26	8 1/4	0.455	7/16	1/4	3.76	10.3	10 1/4	0.760	3/4	1.16	1 5/8	
×33.5[c]	9.84	8.17	8 1/8	0.395	3/8	3/16	3.23	10.2	10 1/4	0.665	11/16	1.07	1 9/16	
WT8×28.5[c]	8.39	8.22	8 1/4	0.430	7/16	1/4	3.53	7.12	7 1/8	0.715	11/16	1.12	1 3/8	3 1/2[g]
×25[c]	7.37	8.13	8 1/8	0.380	3/8	3/16	3.09	7.07	7 1/8	0.630	5/8	1.03	1 5/16	↓
×22.5[c]	6.63	8.07	8 1/8	0.345	3/8	3/16	2.78	7.04	7	0.565	9/16	0.967	1 1/4	
×20[c,h]	5.89	8.01	8	0.305	5/16	3/16	2.44	7.00	7	0.505	1/2	0.907	1 3/16	3 1/2
×18[c,h]	5.29	7.93	7 7/8	0.295	5/16	3/16	2.34	6.99	7	0.430	7/16	0.832	1 1/8	3 1/2
WT8×15.5[c]	4.56	7.94	8	0.275	1/4	1/8	2.18	5.53	5 1/2	0.440	7/16	0.842	1 1/8	3 1/2
×13[c,v]	3.84	7.85	7 7/8	0.250	1/4	1/8	1.96	5.50	5 1/2	0.345	3/8	0.747	1 1/16	3 1/2

[c] Shape is slender for compression with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[h] Flange thickness greater than 2 in. Special requirements may apply per Specification Section A3.1c.

[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1–8 (continued)
WT Shapes
Properties

WT9-WT8

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 50$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
35.5	4.71	18.7	78.2	11.2	2.74	2.26	20.0	0.683	30.1	7.89	1.70	12.3	0.965	1.74	3.96
32.5	5.06	20.4	70.7	10.1	2.72	2.20	18.0	0.629	27.4	7.22	1.69	11.2	0.877	1.36	3.01
30	5.44	22.0	64.7	9.29	2.71	2.16	16.5	0.583	25.0	6.63	1.68	10.3	0.797	1.08	2.35
27.5	5.98	23.2	59.5	8.63	2.71	2.16	15.3	0.538	22.5	5.97	1.67	9.26	0.734	0.830	1.84
25	6.57	25.3	53.5	7.79	2.70	2.12	13.8	0.489	20.0	5.35	1.65	8.28	0.623	0.619	1.36
23	5.01	25.1	52.1	7.77	2.77	2.33	13.9	0.558	11.3	3.71	1.29	5.84	0.636	0.609	1.20
20	5.73	28.4	44.8	6.73	2.76	2.29	12.0	0.489	9.55	3.17	1.27	4.97	0.495	0.404	0.788
17.5	7.06	29.5	40.1	6.21	2.79	2.39	11.2	0.450	7.67	2.56	1.22	4.02	0.460	0.252	0.598
50	5.29	14.5	76.8	11.4	2.28	1.76	20.7	0.706	93.1	17.9	2.51	27.4	1.00	3.85	10.4
44.5	5.92	16.0	67.2	10.1	2.27	1.70	18.1	0.631	81.3	15.7	2.49	24.0	1.00	2.72	7.19
38.5	6.77	18.2	56.9	8.59	2.24	1.63	15.3	0.549	69.2	13.4	2.47	20.5	0.990	1.78	4.61
33.5	7.70	20.7	48.6	7.36	2.22	1.56	13.0	0.481	59.5	11.6	2.46	17.7	0.863	1.19	3.01
28.5	4.98	19.1	48.7	7.77	2.41	1.94	13.8	0.589	21.6	6.06	1.60	9.42	0.942	1.10	1.99
25	5.61	21.4	42.3	6.78	2.40	1.89	12.0	0.521	18.6	5.26	1.59	8.15	0.826	0.760	1.34
22.5	6.23	23.4	37.8	6.10	2.39	1.86	10.8	0.471	16.4	4.67	1.57	7.22	0.726	0.555	0.974
20	6.93	26.2	33.1	5.35	2.37	1.81	9.43	0.421	14.4	4.12	1.56	6.36	0.581	0.396	0.673
18	8.12	26.9	30.6	5.05	2.41	1.88	8.93	0.378	12.2	3.50	1.52	5.42	0.554	0.272	0.516
15.5	6.28	28.9	27.5	4.64	2.45	2.02	8.27	0.413	6.20	2.24	1.17	3.51	0.480	0.230	0.366
13	7.97	31.4	23.5	4.09	2.47	2.09	7.36	0.372	4.79	1.74	1.12	2.73	0.406	0.130	0.243

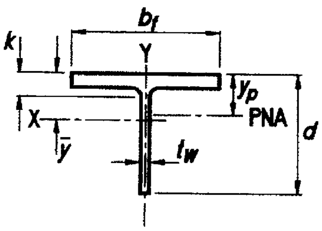


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, *A*	Depth, *d*		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		*k*		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT7×365[h]	107	11.2	11 1/4	3.07	3 1/16	1 9/16	34.4	17.9	17 7/8	4.91	4 15/16	5.51	6 3/16	7 1/2[g]
×332.5[h]	97.8	10.8	10 7/8	2.83	2 13/16	1 7/16	30.6	17.7	17 5/8	4.52	4 1/2	5.12	5 13/16	7 1/2[g]
×302.5[h]	88.9	10.5	10 1/2	2.60	2 5/8	1 5/16	27.1	17.4	17 3/8	4.16	4 3/16	4.76	5 7/16	7 1/2
×275[h]	80.9	10.1	10 1/8	2.38	2 3/8	1 3/16	24.1	17.2	17 1/4	3.82	3 13/16	4.42	5 1/8	↓
×250[h]	73.5	9.80	9 3/4	2.19	2 3/16	1 1/8	21.5	17.0	17	3.50	3 1/2	4.10	4 13/16	↓
×227.5[h]	66.9	9.51	9 1/2	2.02	2	1	19.2	16.8	16 7/8	3.21	3 3/16	3.81	4 1/2	↓
×213[h]	62.6	9.34	9 3/8	1.88	1 7/8	15/16	17.5	16.7	16 3/4	3.04	3 1/16	3.63	4 5/16	↓
×199[h]	58.5	9.15	9 1/8	1.77	1 3/4	7/8	16.2	16.6	16 5/8	2.85	2 7/8	3.44	4 1/8	↓
×185[h]	54.4	8.96	9	1.66	1 5/8	13/16	14.8	16.5	16 1/2	2.66	2 11/16	3.26	3 15/16	↓
×171[h]	50.3	8.77	8 3/4	1.54	1 9/16	13/16	13.5	16.4	16 3/8	2.47	2 1/2	3.07	3 3/4	↓
×155.5[h]	45.7	8.56	8 1/2	1.41	1 7/16	3/4	12.1	16.2	16 1/4	2.26	2 1/4	2.86	3 9/16	↓
×141.5[h]	41.6	8.37	8 3/8	1.29	1 5/16	11/16	10.8	16.1	16 1/8	2.07	2 1/16	2.67	3 3/8	↓
×128.5	37.8	8.19	8 1/4	1.18	1 3/16	5/8	9.62	16.0	16	1.89	1 7/8	2.49	3 3/16	↓
×116.5	34.2	8.02	8	1.07	1 1/16	9/16	8.58	15.9	15 7/8	1.72	1 3/4	2.32	3	↓
×105.5	31.0	7.86	7 7/8	0.980	1	1/2	7.70	15.8	15 3/4	1.56	1 9/16	2.16	2 7/8	↓
×96.5	28.4	7.74	7 3/4	0.890	7/8	7/16	6.89	15.7	15 3/4	1.44	1 7/16	2.04	2 3/4	↓
×88	25.9	7.61	7 5/8	0.830	13/16	7/16	6.32	15.7	15 5/8	1.31	1 5/16	1.91	2 5/8	↓
×79.5	23.4	7.49	7 1/2	0.745	3/4	3/8	5.58	15.6	15 5/8	1.19	1 3/16	1.79	2 1/2	↓
×72.5	21.3	7.39	7 3/8	0.680	11/16	3/8	5.03	15.5	15 1/2	1.09	1 1/16	1.69	2 3/8	↓
WT7×66	19.4	7.33	7 3/8	0.645	5/8	5/16	4.73	14.7	14 3/4	1.03	1	1.63	2 5/16	5 1/2
×60	17.7	7.24	7 1/4	0.590	9/16	5/16	4.27	14.7	14 5/8	0.940	15/16	1.54	2 1/4	↓
×54.5	16.0	7.16	7 1/8	0.525	1/2	1/4	3.76	14.6	14 5/8	0.860	7/8	1.46	2 3/16	↓
×49.5[f]	14.6	7.08	7 1/8	0.485	1/2	1/4	3.43	14.6	14 5/8	0.780	3/4	1.38	2 1/16	↓
×45[f]	13.2	7.01	7	0.440	7/16	1/4	3.08	14.5	14 1/2	0.710	11/16	1.31	2	↓
WT7×41	12.0	7.16	7 1/8	0.510	1/2	1/4	3.65	10.1	10 1/8	0.855	7/8	1.45	1 11/16	5 1/2
×37	10.9	7.09	7 1/8	0.450	7/16	1/4	3.19	10.1	10 1/8	0.785	13/16	1.38	1 5/8	↓
×34	9.99	7.02	7	0.415	7/16	1/4	2.91	10.0	10	0.720	3/4	1.31	1 9/16	↓
×30.5[c]	8.96	6.95	7	0.375	3/8	3/16	2.60	10.0	10	0.645	5/8	1.24	1 1/2	↓
WT7×26.5[c]	7.80	6.96	7	0.370	3/8	3/16	2.58	8.06	8	0.660	11/16	1.25	1 1/2	5 1/2
×24[c]	7.07	6.90	6 7/8	0.340	5/16	3/16	2.34	8.03	8	0.595	5/8	1.19	1 7/16	↓
×21.5[c]	6.31	6.83	6 7/8	0.305	5/16	3/16	2.08	8.00	8	0.530	1/2	1.12	1 3/8	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per Specification Section A3.1c.

Table 1–8 (continued)
WT Shapes
Properties

WT7

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 50$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
365	1.82	3.65	739	95.4	2.62	3.47	211	3.00	2360	264	4.69	408	1.00	714	5250
332.5	1.95	3.82	622	82.1	2.52	3.25	182	2.77	2080	236	4.62	365	1.00	555	3920
302.5	2.09	4.03	524	70.6	2.43	3.05	157	2.55	1840	211	4.55	326	1.00	430	2930
275	2.25	4.25	442	60.9	2.34	2.85	136	2.35	1630	189	4.49	292	1.00	331	2180
250	2.43	4.47	375	52.7	2.26	2.67	117	2.16	1440	169	4.43	261	1.00	254	1620
227.5	2.62	4.72	321	45.9	2.19	2.51	102	1.99	1280	152	4.38	234	1.00	196	1210
213	2.75	4.98	287	41.4	2.14	2.40	91.7	1.88	1180	141	4.34	217	1.00	164	991
199	2.92	5.17	257	37.6	2.10	2.30	82.9	1.76	1090	131	4.31	201	1.00	135	801
185	3.10	5.41	229	33.9	2.05	2.19	74.4	1.65	994	121	4.27	185	1.00	110	640
171	3.31	5.69	203	30.4	2.01	2.09	66.2	1.54	903	110	4.24	169	1.00	88.3	502
155.5	3.59	6.07	176	26.7	1.96	1.97	57.7	1.41	807	99.4	4.20	152	1.00	67.5	375
141.5	3.89	6.49	153	23.5	1.92	1.86	50.4	1.29	722	89.7	4.17	137	1.00	51.8	281
128.5	4.23	6.97	133	20.7	1.88	1.75	43.9	1.18	645	80.7	4.13	123	1.00	39.3	209
116.5	4.62	7.50	116	18.2	1.84	1.65	38.2	1.08	576	72.5	4.10	110	1.00	29.6	154
105.5	5.06	8.02	102	16.2	1.81	1.57	33.4	0.980	513	65.0	4.07	98.9	1.00	22.2	113
96.5	5.45	8.70	89.8	14.4	1.78	1.49	29.4	0.903	466	59.3	4.05	90.1	1.00	17.3	87.2
88	5.97	9.17	80.5	13.0	1.76	1.43	26.3	0.827	419	53.5	4.02	81.3	1.00	13.2	65.2
79.5	6.54	10.1	70.2	11.4	1.73	1.35	22.8	0.751	374	48.1	4.00	73.0	1.00	9.84	47.9
72.5	7.11	10.9	62.5	10.2	1.71	1.29	20.2	0.688	338	43.7	3.98	66.2	1.00	7.56	36.3
66	7.15	11.4	57.8	9.57	1.73	1.29	18.6	0.658	274	37.2	3.76	56.5	1.00	6.13	26.6
60	7.80	12.3	51.7	8.61	1.71	1.24	16.5	0.602	247	33.7	3.74	51.2	1.00	4.67	20.0
54.5	8.49	13.6	45.3	7.56	1.68	1.17	14.4	0.548	223	30.6	3.73	46.3	1.00	3.55	15.0
49.5	9.34	14.6	40.9	6.88	1.67	1.14	12.9	0.500	201	27.6	3.71	41.8	—	2.68	11.1
45	10.2	15.9	36.5	6.16	1.66	1.09	11.5	0.456	181	25.0	3.70	37.8	1.00	2.03	8.31
41	5.92	14.0	41.2	7.14	1.85	1.39	13.2	0.593	74.1	14.6	2.48	22.4	1.00	2.53	5.63
37	6.41	15.7	36.0	6.25	1.82	1.32	11.5	0.541	66.9	13.3	2.48	20.2	1.00	1.93	4.19
34	6.97	16.9	32.6	5.69	1.81	1.29	10.4	0.498	60.7	12.1	2.46	18.4	1.00	1.50	3.21
30.5	7.75	18.5	28.9	5.07	1.80	1.25	9.15	0.448	53.7	10.7	2.45	16.4	0.972	1.09	2.29
26.5	6.11	18.8	27.6	4.94	1.88	1.38	8.87	0.484	28.8	7.15	1.92	11.0	0.957	0.967	1.46
24	6.75	20.3	24.9	4.49	1.88	1.35	8.00	0.440	25.7	6.40	1.91	9.80	0.883	0.723	1.07
21.5	7.54	22.4	21.9	3.98	1.86	1.31	7.05	0.395	22.6	5.65	1.89	8.64	0.776	0.522	0.751

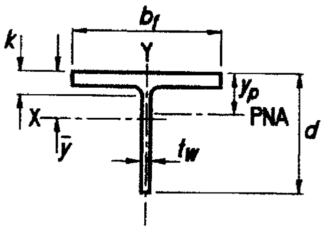


Table 1-8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT7×19[c]	5.58	7.05	7	0.310	5/16	3/16	2.19	6.77	6 3/4	0.515	1/2	0.915	1 1/4	3 1/2[g]
×17[c]	5.00	6.99	7	0.285	5/16	3/16	1.99	6.75	6 3/4	0.455	7/16	0.855	1 3/16	3 1/2
×15[c]	4.42	6.92	6 7/8	0.270	1/4	1/8	1.87	6.73	6 3/4	0.385	3/8	0.785	1 1/8	3 1/2
WT7×13[c]	3.85	6.96	7	0.255	1/4	1/8	1.77	5.03	5	0.420	7/16	0.820	1 1/8	2 3/4[g]
×11[c,v]	3.25	6.87	6 7/8	0.230	1/4	1/8	1.58	5.00	5	0.335	5/16	0.735	1 1/16	2 3/4[g]
WT6×168[h]	49.4	8.41	8 3/8	1.78	1 3/4	7/8	14.9	13.4	13 3/8	2.96	2 15/16	3.55	3 7/8	5 1/2
×152.5[h]	44.8	8.16	8 1/8	1.63	1 5/8	13/16	13.3	13.2	13 1/4	2.71	2 11/16	3.30	3 5/8	
×139.5[h]	41.0	7.93	7 7/8	1.53	1 1/2	3/4	12.1	13.1	13 1/8	2.47	2 1/2	3.07	3 3/8	
×126[h]	37.0	7.71	7 3/4	1.40	1 3/8	11/16	10.7	13.0	13	2.25	2 1/4	2.85	3 1/8	
×115[h]	33.9	7.53	7 1/2	1.29	1 5/16	11/16	9.67	12.9	12 7/8	2.07	2 1/16	2.67	2 15/16	
×105	30.9	7.36	7 3/8	1.18	1 3/16	5/8	8.68	12.8	12 3/4	1.90	1 7/8	2.50	2 13/16	
×95	27.9	7.19	7 1/4	1.06	1 1/16	9/16	7.62	12.7	12 5/8	1.74	1 3/4	2.33	2 5/8	
×85	25.0	7.02	7	0.960	15/16	1/2	6.73	12.6	12 5/8	1.56	1 9/16	2.16	2 7/16	
×76	22.4	6.86	6 7/8	0.870	7/8	7/16	5.96	12.5	12 1/2	1.40	1 3/8	2.00	2 5/16	
×68	20.0	6.71	6 3/4	0.790	13/16	7/16	5.30	12.4	12 3/8	1.25	1 1/4	1.85	2 1/8	
×60	17.6	6.56	6 1/2	0.710	11/16	3/8	4.66	12.3	12 3/8	1.11	1 1/8	1.70	2	
×53	15.6	6.45	6 1/2	0.610	5/8	5/16	3.93	12.2	12 1/4	0.990	1	1.59	1 7/8	
×48	14.1	6.36	6 3/8	0.550	9/16	5/16	3.50	12.2	12 1/8	0.900	7/8	1.50	1 13/16	
×43.5	12.8	6.27	6 1/4	0.515	1/2	1/4	3.23	12.1	12 1/8	0.810	13/16	1.41	1 11/16	
×39.5	11.6	6.19	6 1/4	0.470	1/2	1/4	2.91	12.1	12 1/8	0.735	3/4	1.33	1 5/8	
×36	10.6	6.13	6 1/8	0.430	7/16	1/4	2.63	12.0	12	0.670	11/16	1.27	1 9/16	
×32.5[f]	9.54	6.06	6	0.390	3/8	3/16	2.36	12.0	12	0.605	5/8	1.20	1 1/2	↓
WT6×29	8.52	6.10	6 1/8	0.360	3/8	3/16	2.19	10.0	10	0.640	5/8	1.24	1 1/2	5 1/2
×26.5	7.78	6.03	6	0.345	3/8	3/16	2.08	10.0	10	0.575	9/16	1.18	1 3/8	5 1/2
WT6×25	7.30	6.10	6 1/8	0.370	3/8	3/16	2.26	8.08	8 1/8	0.640	5/8	1.14	1 1/2	5 1/2
×22.5	6.56	6.03	6	0.335	5/16	3/16	2.02	8.05	8	0.575	9/16	1.08	1 3/8	
×20[c]	5.84	5.97	6	0.295	5/16	3/16	1.76	8.01	8	0.515	1/2	1.02	1 3/8	↓
WT6×17.5[c]	5.17	6.25	6 1/4	0.300	5/16	3/16	1.88	6.56	6 1/2	0.520	1/2	0.820	1 3/16	3 1/2
×15[c]	4.40	6.17	6 1/8	0.260	1/4	1/8	1.60	6.52	6 1/2	0.440	7/16	0.740	1 1/8	
×13[c]	3.82	6.11	6 1/8	0.230	1/4	1/8	1.41	6.49	6 1/2	0.380	3/8	0.680	1 1/16	↓

[c] Shape is slender for compression with $F_y = 50$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[h] Flange thickness greater than 2 in. Special requirements may apply per Specification Section A3.1c.
[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1–8 (continued)
WT Shapes
Properties

WT7-WT6

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 50$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
19	6.57	22.7	23.3	4.22	2.04	1.54	7.45	0.412	13.3	3.94	1.55	6.07	0.758	0.398	0.554
17	7.41	24.5	20.9	3.83	2.04	1.53	6.74	0.371	11.6	3.45	1.53	5.32	0.668	0.284	0.400
15	8.74	25.6	19.0	3.55	2.07	1.58	6.25	0.329	9.79	2.91	1.49	4.49	0.609	0.190	0.287
13	5.98	27.3	17.3	3.31	2.12	1.72	5.89	0.383	4.45	1.77	1.08	2.76	0.538	0.179	0.207
11	7.46	29.9	14.8	2.91	2.14	1.76	5.20	0.325	3.50	1.40	1.04	2.19	0.448	0.104	0.134
168	2.26	4.74	190	31.2	1.96	2.31	68.4	1.84	593	88.6	3.47	137	1.00	120	481
152.5	2.45	5.02	162	27.0	1.90	2.16	59.1	1.69	525	79.3	3.42	122	1.00	92.0	356
139.5	2.66	5.18	141	24.1	1.86	2.05	51.9	1.56	469	71.3	3.38	110	1.00	70.9	267
126	2.89	5.52	121	20.9	1.81	1.92	44.8	1.42	414	63.6	3.34	97.9	1.00	53.5	195
115	3.11	5.86	106	18.5	1.77	1.82	39.4	1.31	371	57.5	3.31	88.4	1.00	41.6	148
105	3.37	6.23	92.1	16.4	1.73	1.72	34.5	1.21	332	51.9	3.28	79.7	1.00	32.1	112
95	3.65	6.78	79.0	14.2	1.68	1.62	29.8	1.10	295	46.5	3.25	71.2	1.00	24.3	82.1
85	4.03	7.31	67.8	12.3	1.65	1.52	25.6	0.994	259	41.2	3.22	62.9	1.00	17.7	58.3
76	4.46	7.88	58.5	10.8	1.62	1.43	22.0	0.896	227	36.4	3.19	55.6	1.00	12.8	41.3
68	4.96	8.49	50.6	9.46	1.59	1.35	19.0	0.805	199	32.1	3.16	48.9	1.00	9.21	28.9
60	5.57	9.24	43.4	8.22	1.57	1.28	16.2	0.716	172	28.0	3.13	42.7	1.00	6.42	19.7
53	6.17	10.6	36.3	6.92	1.53	1.19	13.6	0.637	151	24.7	3.11	37.5	1.00	4.55	13.6
48	6.76	11.6	32.0	6.12	1.51	1.13	11.9	0.580	135	22.2	3.09	33.7	1.00	3.42	10.1
43.5	7.48	12.2	28.9	5.60	1.50	1.10	10.7	0.527	120	19.9	3.07	30.2	1.00	2.54	7.34
39.5	8.22	13.2	25.8	5.03	1.49	1.06	9.49	0.480	108	17.9	3.05	27.1	1.00	1.91	5.43
36	8.99	14.2	23.2	4.54	1.48	1.02	8.48	0.439	97.5	16.2	3.04	24.6	1.00	1.46	4.07
32.5	9.92	15.5	20.6	4.06	1.47	0.985	7.50	0.398	87.2	14.5	3.02	22.0	1.00	1.09	2.97
29	7.82	16.9	19.1	3.76	1.50	1.03	6.97	0.426	53.5	10.7	2.51	16.2	1.00	1.05	2.08
26.5	8.69	17.5	17.7	3.54	1.51	1.02	6.46	0.389	47.9	9.58	2.48	14.5	1.00	0.788	1.53
25	6.31	16.5	18.7	3.79	1.60	1.17	6.88	0.452	28.2	6.97	1.96	10.6	1.00	0.855	1.23
22.5	7.00	18.0	16.6	3.39	1.59	1.13	6.10	0.408	25.0	6.21	1.95	9.47	0.998	0.627	0.885
20	7.77	20.2	14.4	2.95	1.57	1.09	5.28	0.365	22.0	5.50	1.94	8.38	0.885	0.452	0.620
17.5	6.31	20.8	16.0	3.23	1.76	1.30	5.71	0.394	12.2	3.73	1.54	5.73	0.855	0.369	0.437
15	7.41	23.7	13.5	2.75	1.75	1.27	4.83	0.337	10.2	3.12	1.52	4.78	0.708	0.228	0.267
13	8.54	26.6	11.7	2.40	1.75	1.25	4.20	0.295	8.66	2.67	1.51	4.08	0.567	0.150	0.174

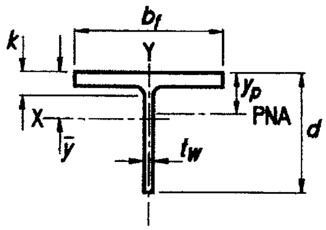


Table 1-8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT6×11^c	3.24	6.16	6 1/8	0.260	1/4	1/8	1.60	4.03	4	0.425	7/16	0.725	15/16	2 1/4^g
×9.5^c	2.79	6.08	6 1/8	0.235	1/4	1/8	1.43	4.01	4	0.350	3/8	0.650	7/8	↓
×8^c	2.36	6.00	6	0.220	1/4	1/8	1.32	3.99	4	0.265	1/4	0.565	13/16	↓
×7^{c,v}	2.08	5.96	6	0.200	3/16	1/8	1.19	3.97	4	0.225	1/4	0.525	3/4	↓
WT5×56	16.5	5.68	5 5/8	0.755	3/4	3/8	4.29	10.4	10 3/8	1.25	1 1/4	1.75	1 15/16	5 1/2
×50	14.7	5.55	5 1/2	0.680	11/16	3/8	3.77	10.3	10 3/8	1.12	1 1/8	1.62	1 13/16	↓
×44	12.9	5.42	5 3/8	0.605	5/8	5/16	3.28	10.3	10 1/4	0.990	1	1.49	1 11/16	↓
×38.5	11.3	5.30	5 1/4	0.530	1/2	1/4	2.81	10.2	10 1/4	0.870	7/8	1.37	1 9/16	↓
×34	9.99	5.20	5 1/4	0.470	1/2	1/4	2.44	10.1	10 1/8	0.770	3/4	1.27	1 7/16	↓
×30	8.82	5.11	5 1/8	0.420	7/16	1/4	2.15	10.1	10 1/8	0.680	11/16	1.18	1 3/8	↓
×27	7.91	5.05	5	0.370	3/8	3/16	1.87	10.0	10	0.615	5/8	1.12	1 5/16	↓
×24.5	7.21	4.99	5	0.340	5/16	3/16	1.70	10.0	10	0.560	9/16	1.06	1 1/4	↓
WT5×22.5	6.63	5.05	5	0.350	3/8	3/16	1.77	8.02	8	0.620	5/8	1.12	1 5/16	↓
×19.5	5.73	4.96	5	0.315	5/16	3/16	1.56	7.99	8	0.530	1/2	1.03	1 3/16	↓
×16.5	4.85	4.87	4 7/8	0.290	5/16	3/16	1.41	7.96	8	0.435	7/16	0.935	1 1/8	↓
WT5×15	4.42	5.24	5 1/4	0.300	5/16	3/16	1.57	5.81	5 3/4	0.510	1/2	0.810	1 1/8	2 3/4^g
×13^c	3.81	5.17	5 1/8	0.260	1/4	1/8	1.34	5.77	5 3/4	0.440	7/16	0.740	1 1/16	↓
×11^c	3.24	5.09	5 1/8	0.240	1/4	1/8	1.22	5.75	5 3/4	0.360	3/8	0.660	15/16	↓
WT5×9.5^c	2.81	5.12	5 1/8	0.250	1/4	1/8	1.28	4.02	4	0.395	3/8	0.695	15/16	2 1/4^g
×8.5^c	2.50	5.06	5	0.240	1/4	1/8	1.21	4.01	4	0.330	5/16	0.630	7/8	↓
×7.5^c	2.21	5.00	5	0.230	1/4	1/8	1.15	4.00	4	0.270	1/4	0.570	13/16	↓
×6^{c,f}	1.77	4.94	4 7/8	0.190	3/16	1/8	0.938	3.96	4	0.210	3/16	0.510	3/4	↓
WT4×33.5	9.84	4.50	4 1/2	0.570	9/16	5/16	2.57	8.28	8 1/4	0.935	15/16	1.33	1 5/8	5 1/2
×29	8.54	4.38	4 3/8	0.510	1/2	1/4	2.23	8.22	8 1/4	0.810	13/16	1.20	1 1/2	↓
×24	7.05	4.25	4 1/4	0.400	3/8	3/16	1.70	8.11	8 1/8	0.685	11/16	1.08	1 3/8	↓
×20	5.87	4.13	4 1/8	0.360	3/8	3/16	1.49	8.07	8 1/8	0.560	9/16	0.954	1 1/4	↓
×17.5	5.14	4.06	4	0.310	5/16	3/16	1.26	8.02	8	0.495	1/2	0.889	1 3/16	↓
×15.5^f	4.56	4.00	4	0.285	5/16	3/16	1.14	8.00	8	0.435	7/16	0.829	1 1/8	↓
WT4×14	4.12	4.03	4	0.285	5/16	3/16	1.15	6.54	6 1/2	0.465	7/16	0.859	15/16	3 1/2
×12	3.54	3.97	4	0.245	1/4	1/8	0.971	6.50	6 1/2	0.400	3/8	0.794	7/8	3 1/2

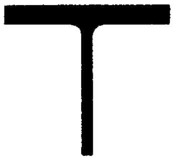
[c] Shape is slender for compression with $F_y = 50$ ksi.

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

[v] Shear strength controlled by buckling effects ($C_v < 1.0$) with $F_y = 50$ ksi.

Table 1–8 (continued)
WT Shapes
Properties



WT6-WT4

Nominal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 50$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
11	4.74	23.7	11.7	2.59	1.90	1.63	4.63	0.402	2.33	1.15	0.847	1.83	0.711	0.146	0.137
9.5	5.72	25.9	10.1	2.28	1.90	1.65	4.11	0.348	1.88	0.939	0.821	1.49	0.598	0.0899	0.0934
8	7.53	27.3	8.70	2.04	1.92	1.74	3.72	0.639	1.41	0.706	0.773	1.13	0.539	0.0511	0.0678
7	8.82	29.8	7.67	1.83	1.92	1.76	3.32	0.760	1.18	0.593	0.753	0.947	0.451	0.0350	0.0493
56	4.17	7.52	28.6	6.40	1.32	1.21	13.4	0.791	118	22.6	2.67	34.6	1.00	7.50	16.9
50	4.62	8.16	24.5	5.56	1.29	1.13	11.4	0.711	103	20.0	2.65	30.5	1.00	5.41	11.9
44	5.18	8.96	20.8	4.77	1.27	1.06	9.65	0.631	89.3	17.4	2.63	26.5	1.00	3.75	8.02
38.5	5.86	10.0	17.4	4.05	1.24	0.990	8.06	0.555	76.8	15.1	2.60	22.9	1.00	2.55	5.31
34	6.58	11.1	14.9	3.49	1.22	0.932	6.85	0.493	66.7	13.2	2.58	20.0	1.00	1.78	3.62
30	7.41	12.2	12.9	3.04	1.21	0.884	5.87	0.438	58.1	11.5	2.57	17.5	1.00	1.23	2.46
27	8.15	13.6	11.1	2.64	1.19	0.836	5.05	0.395	51.7	10.3	2.56	15.6	1.00	0.909	1.78
24.5	8.93	14.7	10.0	2.39	1.18	0.807	4.52	0.361	46.7	9.34	2.54	14.1	1.00	0.693	1.33
22.5	6.47	14.4	10.2	2.47	1.24	0.907	4.65	0.413	26.7	6.65	2.01	10.1	1.00	0.753	0.981
19.5	7.53	15.7	8.84	2.16	1.24	0.876	3.99	0.359	22.5	5.64	1.98	8.57	1.00	0.487	0.616
16.5	9.15	16.8	7.71	1.93	1.26	0.869	3.48	0.305	18.3	4.60	1.94	7.00	1.00	0.291	0.356
15	5.70	17.5	9.28	2.24	1.45	1.10	4.01	0.380	8.35	2.87	1.37	4.41	1.00	0.310	0.273
13	6.56	19.9	7.86	1.91	1.44	1.06	3.39	0.330	7.05	2.44	1.36	3.75	0.904	0.201	0.173
11	7.99	21.2	6.88	1.72	1.46	1.07	3.02	0.282	5.71	1.99	1.33	3.05	0.837	0.119	0.107
9.5	5.09	20.5	6.68	1.74	1.54	1.28	3.10	0.349	2.15	1.07	0.874	1.67	0.873	0.116	0.0796
8.5	6.08	21.1	6.06	1.62	1.56	1.32	2.90	0.311	1.78	0.887	0.844	1.40	0.843	0.0776	0.0610
7.5	7.41	21.7	5.45	1.50	1.57	1.37	2.71	0.305	1.45	0.723	0.810	1.15	0.810	0.0518	0.0475
6	9.43	26.0	4.35	1.22	1.57	1.36	2.20	0.322	1.09	0.551	0.785	0.869	0.593	0.0272	0.0255
33.5	4.43	7.89	10.9	3.05	1.05	0.936	6.29	0.594	44.3	10.7	2.12	16.3	1.00	2.51	3.56
29	5.07	8.58	9.12	2.61	1.03	0.874	5.25	0.520	37.5	9.13	2.10	13.9	1.00	1.66	2.28
24	5.92	10.6	6.85	1.97	0.986	0.777	3.94	0.435	30.5	7.51	2.08	11.4	1.00	0.977	1.30
20	7.21	11.5	5.73	1.69	0.988	0.735	3.25	0.364	24.5	6.08	2.04	9.24	1.00	0.558	0.715
17.5	8.10	13.1	4.82	1.43	0.968	0.688	2.71	0.321	21.3	5.31	2.03	8.05	1.00	0.384	0.480
15.5	9.19	14.0	4.28	1.28	0.969	0.668	2.39	0.285	18.5	4.64	2.02	7.03	1.00	0.267	0.327
14	7.03	14.1	4.23	1.28	1.01	0.734	2.38	0.315	10.8	3.31	1.62	5.04	1.00	0.268	0.230
12	8.12	16.2	3.53	1.08	0.999	0.695	1.98	0.272	9.14	2.81	1.61	4.28	1.00	0.173	0.144

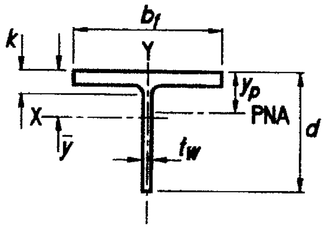


Table 1–8 (continued)
WT Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance		
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k		Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.	in.
WT4×10.5	3.08	4.14	4 1/8	0.250	1/4	1/8	1.04	5.27	5 1/4	0.400	3/8	0.700	7/8	2 3/4[g]
×9	2.63	4.07	4 1/8	0.230	1/4	1/8	0.936	5.25	5 1/4	0.330	5/16	0.630	13/16	2 3/4[g]
WT4×7.5	2.22	4.06	4	0.245	1/4	1/8	0.993	4.02	4	0.315	5/16	0.615	13/16	2 1/4[g]
×6.5	1.92	4.00	4	0.230	1/4	1/8	0.919	4.00	4	0.255	1/4	0.555	3/4	↓
×5[c,f]	1.48	3.95	4	0.170	3/16	1/8	0.671	3.94	4	0.205	3/16	0.505	11/16	↓
WT3×12.5	3.67	3.19	3 1/4	0.320	5/16	3/16	1.02	6.08	6 1/8	0.455	7/16	0.705	15/16	3 1/2
×10	2.94	3.10	3 1/8	0.260	1/4	1/8	0.806	6.02	6	0.365	3/8	0.615	7/8	↓
×7.5[f]	2.21	3.00	3	0.230	1/4	1/8	0.689	5.99	6	0.260	1/4	0.510	3/4	↓
WT3×8	2.37	3.14	3 1/8	0.260	1/4	1/8	0.816	4.03	4	0.405	3/8	0.655	7/8	2 1/4[g]
×6	1.78	3.02	3	0.230	1/4	1/8	0.693	4.00	4	0.280	1/4	0.530	3/4	↓
×4.5[f]	1.34	2.95	3	0.170	3/16	1/8	0.502	3.94	4	0.215	3/16	0.465	11/16	↓
×4.25[f]	1.26	2.92	2 7/8	0.170	3/16	1/8	0.496	3.94	4	0.195	3/16	0.445	11/16	↓
WT2.5×9.5	2.78	2.58	2 5/8	0.270	1/4	1/8	0.695	5.03	5	0.430	7/16	0.730	13/16	2 3/4
×8	2.35	2.51	2 1/2	0.240	1/4	1/8	0.601	5.00	5	0.360	3/8	0.660	3/4	2 3/4
WT2×6.5	1.91	2.08	2 1/8	0.280	1/4	1/8	0.582	4.06	4	0.345	3/8	0.595	3/4	2 1/4

[c] Shape is slender for compression with $F_y = 50$ ksi.

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

Table 1–8 (continued)
WT Shapes
Properties

WT4-WT2

Nominal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	F_y = 50 ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
10.5	6.59	16.6	3.90	1.18	1.12	0.831	2.11	0.292	4.88	1.85	1.26	2.84	1.00	0.141	0.0916
9	7.95	17.7	3.41	1.05	1.14	0.834	1.86	0.251	3.98	1.52	1.23	2.33	1.00	0.0855	0.0562
7.5	6.37	16.6	3.28	1.07	1.22	0.998	1.91	0.276	1.70	0.849	0.876	1.33	1.00	0.0679	0.0382
6.5	7.84	17.4	2.89	0.974	1.23	1.03	1.74	0.240	1.36	0.682	0.843	1.07	1.00	0.0433	0.0269
5	9.61	23.2	2.15	0.717	1.20	0.953	1.27	0.188	1.05	0.531	0.840	0.826	0.735	0.0212	0.0114
12.5	6.68	10.0	2.29	0.886	0.789	0.610	1.68	0.302	8.53	2.81	1.52	4.28	1.00	0.229	0.171
10	8.25	11.9	1.76	0.693	0.774	0.560	1.29	0.244	6.64	2.21	1.50	3.36	1.00	0.120	0.0858
7.5	11.5	13.0	1.41	0.577	0.797	0.558	1.03	0.185	4.66	1.56	1.45	2.37	1.00	0.0504	0.0342
8	4.98	12.1	1.69	0.685	0.844	0.676	1.25	0.294	2.21	1.10	0.966	1.69	1.00	0.111	0.0426
6	7.14	13.1	1.32	0.564	0.862	0.677	1.01	0.222	1.50	0.748	0.918	1.16	1.00	0.0449	0.0178
4.5	9.16	17.4	0.950	0.408	0.842	0.623	0.720	0.170	1.10	0.557	0.905	0.856	1.00	0.0202	0.00736
4.25	10.1	17.1	0.905	0.397	0.848	0.637	0.700	0.160	0.995	0.505	0.890	0.778	1.00	0.0166	0.00620
9.5	5.85	9.54	1.01	0.485	0.604	0.487	0.970	0.276	4.56	1.81	1.28	2.76	1.00	0.157	0.0775
8	6.94	10.4	0.845	0.413	0.599	0.458	0.801	0.235	3.75	1.50	1.26	2.28	1.00	0.0958	0.0453
6.5	5.88	7.43	0.526	0.321	0.524	0.440	0.616	0.236	1.93	0.950	1.00	1.46	1.00	0.0750	0.0233

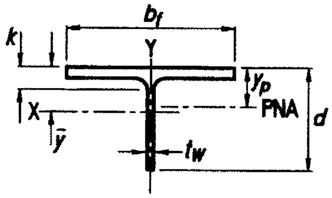


Table 1-9

MT Shapes

Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance	
				Thickness, t_w		$\frac{t_w}{2}$	Area	Width, b_f		Thickness, t_f		k	Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.
MT6.25×6.2^{c,v}	1.80	6.27	6 1/4	0.1550	1/8	1/16	0.971	3.75	3 3/4	0.228	1/4	9/16	—
×5.8^{c,v}	1.69	6.25	6 1/4	0.155	1/8	1/16	0.969	3.50	3 1/2	0.211	3/16	9/16	
MT6×5.9^{c}	1.72	6.00	6	0.177	3/16	1/8	1.06	3.07	3 1/8	0.225	1/4	9/16	—
×5.4^{c,v}	1.58	5.99	6	0.160	3/16	1/8	0.958	3.07	3 1/8	0.210	3/16	9/16	—
×5^{c,v}	1.46	5.99	6	0.149	1/8	1/16	0.892	3.25	3 1/4	0.180	3/16	1/2	—
MT5×4.5^{c}	1.32	5.00	5	0.157	3/16	1/8	0.785	2.69	2 3/4	0.206	3/16	9/16	—
×4^{c}	1.17	4.98	5	0.141	1/8	1/16	0.701	2.69	2 3/4	0.182	3/16	9/16	—
MT5×3.75^{c,v}	1.10	5.00	5	0.130	1/8	1/16	0.649	2.69	2 3/4	0.173	3/16	7/16	—
MT4×3.25^{c,v}	0.953	4.00	4	0.135	1/8	1/16	0.540	2.28	2 1/4	0.189	3/16	9/16	—
×3.1^{c}	0.904	4.00	4	0.129	1/8	1/16	0.516	2.28	2 1/4	0.177	3/16	7/16	—
MT3×2.2^{c}	0.643	3.00	3	0.114	1/8	1/16	0.342	1.84	1 7/8	0.171	3/16	3/8	—
×1.85^{c}	0.540	2.96	3	0.0980	1/8	1/16	0.290	2.00	2	0.129	1/8	5/16	—
MT2.5×9.45^{t}	2.76	2.50	2 1/2	0.316	5/16	3/16	0.790	5.00	5	0.416	7/16	13/16	2 3/4^{g}
MT2×3^{f}	0.855	1.90	1 7/8	0.130	1/8	1/16	0.247	3.80	3 3/4	0.160	3/16	1/2	—

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
[g] The actual size, combination, and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.
[t] This shape has tapered flanges while all other MT-shapes have parallel flange surfaces.
[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 36 ksi.
— Flange is too narrow to establish a workable gage.

Table 1–9 (continued)
MT Shapes
Properties

MT SHAPES

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{h}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 36$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.^{6}
6.2	8.22	40.4	7.29	1.61	2.01	1.74	2.92	0.372	1.00	0.536	0.746	0.839	0.340	0.0246	0.0284
5.8	8.29	40.3	6.94	1.57	2.03	1.84	2.86	0.808	0.756	0.432	0.669	0.684	0.342	0.0206	0.0268
5.9	6.81	33.9	6.61	1.61	1.96	1.89	2.89	1.13	0.543	0.354	0.561	0.575	0.483	0.0249	0.0337
5.4	7.30	37.4	6.03	1.46	1.95	1.86	2.63	1.05	0.506	0.330	0.566	0.532	0.397	0.0196	0.0250
5	9.03	40.2	5.62	1.36	1.96	1.86	2.45	1.08	0.517	0.318	0.594	0.509	0.344	0.0145	0.0202
4.5	6.53	31.8	3.47	1.00	1.62	1.54	1.81	0.808	0.336	0.250	0.505	0.403	0.548	0.0156	0.0138
4	7.39	35.3	3.08	0.894	1.62	1.52	1.61	0.809	0.296	0.220	0.502	0.354	0.446	0.0112	0.00989
3.75	7.77	38.4	2.91	0.836	1.63	1.51	1.51	0.759	0.281	0.209	0.505	0.334	0.376	0.00932	0.00792
3.25	6.03	29.6	1.57	0.558	1.29	1.18	1.01	0.472	0.188	0.165	0.444	0.264	0.633	0.00917	0.00463
3.1	6.44	31.0	1.50	0.533	1.29	1.18	0.967	0.497	0.176	0.154	0.441	0.247	0.578	0.00778	0.00403
2.2	5.39	26.3	0.579	0.268	0.949	0.841	0.483	0.190	0.0897	0.0973	0.374	0.155	0.779	0.00494	0.00124
1.85	7.75	30.2	0.483	0.226	0.945	0.827	0.409	0.174	0.0863	0.0863	0.400	0.136	0.609	0.00265	0.000754
9.45	6.01	7.91	1.05	0.528	0.617	0.512	1.03	0.276	4.35	1.74	1.26	2.66	1.00	0.156	0.0732
3	11.9	14.6	0.208	0.133	0.493	0.341	0.241	0.112	0.732	0.385	0.926	0.588	1.00	0.00919	0.00193

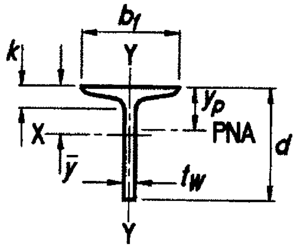


Table 1–10
ST Shapes
Dimensions

Shape	Area, A	Depth, d		Stem				Flange				Distance	
				Thickness, t_w		t_w	Area	Width, b_f		Thickness, t_f		k	Workable Gage
	in.^{2}	in.		in.		in.	in.^{2}	in.		in.		in.	in.
ST12×60.5	17.8	12.3	12 1/4	0.800	13/16	7/16	9.80	8.05	8	1.09	1 1/16	2	4
×53	15.6	12.3	12 1/4	0.620	5/8	5/16	7.60	7.87	7 7/8	1.09	1 1/16	2	4
ST12×50	14.7	12.0	12	0.745	3/4	3/8	8.94	7.25	7 1/4	0.870	7/8	1 3/4	4
×45	13.2	12.0	12	0.625	5/8	5/16	7.50	7.13	7 1/8	0.870	7/8	1 3/4	4
×40[c]	11.7	12.0	12	0.500	1/2	1/4	6.00	7.00	7	0.870	7/8	1 3/4	4
ST10×48	14.1	10.2	10 1/8	0.800	13/16	7/16	8.12	7.20	7 1/4	0.920	15/16	1 3/4	4
×43	12.7	10.2	10 1/8	0.660	11/16	3/8	6.70	7.06	7	0.920	15/16	1 3/4	4
ST10×37.5	11.0	10.0	10	0.635	5/8	5/16	6.35	6.39	6 3/8	0.795	13/16	1 5/8	3 1/2[g]
×33	9.69	10.0	10	0.505	1/2	1/4	5.05	6.26	6 1/4	0.795	13/16	1 5/8	3 1/2[g]
ST9×35	10.3	9.00	9	0.711	11/16	3/8	6.40	6.25	6 1/4	0.691	11/16	1 1/2	3 1/2[g]
×27.35	8.02	9.00	9	0.461	7/16	1/4	4.15	6.00	6	0.691	11/16	1 1/2	3 1/2[g]
ST7.5×25	7.34	7.50	7 1/2	0.550	9/16	5/16	4.13	5.64	5 5/8	0.622	5/8	1 3/8	3 1/2[g]
×21.45	6.30	7.50	7 1/2	0.411	7/16	1/4	3.08	5.50	5 1/2	0.622	5/8	1 3/8	3 1/2[g]
ST6×25	7.32	6.00	6	0.687	11/16	3/8	4.12	5.48	5 1/2	0.659	11/16	1 7/16	3[g]
×20.4	5.96	6.00	6	0.462	7/16	1/4	2.77	5.25	5 1/4	0.659	11/16	1 7/16	3[g]
ST6×17.5	5.12	6.00	6	0.428	7/16	1/4	2.57	5.08	5 1/8	0.544	9/16	1 3/16	3[g]
×15.9	4.65	6.00	6	0.350	3/8	3/16	2.10	5.00	5	0.544	9/16	1 3/16	3[g]
ST5×17.5	5.14	5.00	5	0.594	5/8	5/16	2.97	4.94	5	0.491	1/2	1 1/8	2 3/4[g]
×12.7	3.73	5.00	5	0.311	5/16	3/16	1.56	4.66	4 5/8	0.491	1/2	1 1/8	2 3/4[g]
ST4×11.5	3.38	4.00	4	0.441	7/16	1/4	1.76	4.17	4 1/8	0.425	7/16	1	2 1/4[g]
×9.2	2.70	4.00	4	0.271	1/4	1/8	1.08	4.00	4	0.425	7/16	1	2 1/4[g]
ST3×8.6	2.53	3.00	3	0.465	7/16	1/4	1.40	3.57	3 5/8	0.359	3/8	13/16	—
×6.25	1.83	3.00	3	0.232	1/4	1/8	0.696	3.33	3 3/8	0.359	3/8	13/16	—
ST2.5×5	1.47	2.50	2 1/2	0.214	3/16	1/8	0.535	3.00	3	0.326	5/16	3/4	—
ST2×4.75	1.39	2.00	2	0.326	5/16	3/16	0.652	2.80	2 3/4	0.293	5/16	3/4	—
×3.85	1.13	2.00	2	0.193	3/16	1/8	0.386	2.66	2 5/8	0.293	5/16	3/4	—
ST1.5×3.75	1.10	1.50	1 1/2	0.349	3/8	3/16	0.524	2.51	2 1/2	0.260	1/4	5/8	—
×2.85	0.830	1.50	1 1/2	0.170	3/16	1/8	0.255	2.33	2 3/8	0.260	1/4	5/8	—

[c] Shape is slender for compression with $F_y = 36$ ksi

[g] The actual size, combination and orientation of fastener components should be compared with the geometry of the cross-section to ensure compatibility.

— Flange is too narrow to establish a workable gage.

Table 1–10 (continued)
ST Shapes
Properties

ST SHAPES

Nom-inal Wt.	Compact Section Criteria		Axis X-X						Axis Y-Y				Q_s	Torsional Properties	
	$\frac{b_f}{2t_f}$	$\frac{d}{t_w}$	I	S	r	$\bar{y}$	Z	y_p	I	S	r	Z	$F_y = 36$ ksi	J	C_w
lb/ft			in.^{4}	in.^{3}	in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}		in.^{4}	in.
60.5	3.69	15.3	259	30.1	3.82	3.63	54.5	1.26	41.5	10.3	1.53	18.1	1.00	6.38	27.5
53.0	3.61	19.8	216	24.1	3.72	3.28	43.3	1.02	38.4	9.76	1.57	16.7	1.00	5.05	15.0
50.0	4.16	16.1	215	26.3	3.83	3.84	47.5	2.16	23.7	6.55	1.27	12.0	1.00	3.76	19.5
45.0	4.09	19.2	190	22.6	3.79	3.60	41.1	1.42	22.3	6.27	1.30	11.2	1.00	3.01	12.1
40.0	4.02	24.0	162	18.6	3.72	3.30	33.6	0.909	21.0	6.00	1.34	10.4	0.878	2.44	6.94
48.0	3.91	12.7	143	20.3	3.18	3.13	36.9	1.35	25.0	6.93	1.33	12.5	1.00	4.16	15.0
43.0	3.84	15.4	124	17.2	3.13	2.91	31.1	0.972	23.3	6.59	1.36	11.6	1.00	3.30	9.17
37.5	4.02	15.7	109	15.8	3.15	3.07	28.6	1.34	14.8	4.62	1.16	8.36	1.00	2.28	7.21
33.0	3.93	19.8	92.9	12.9	3.10	2.81	23.4	0.841	13.7	4.39	1.19	7.70	1.00	1.78	4.02
35.0	4.52	12.7	84.5	14.0	2.87	2.94	25.1	1.78	12.0	3.84	1.08	7.17	1.00	2.02	7.03
27.4	4.34	19.5	62.3	9.60	2.79	2.51	17.3	0.737	10.4	3.45	1.14	6.06	1.00	1.16	2.26
25.0	4.53	13.6	40.5	7.72	2.35	2.25	14.0	0.826	7.79	2.76	1.03	4.99	1.00	1.05	2.02
21.5	4.42	18.2	32.9	5.99	2.29	2.01	10.8	0.605	7.13	2.59	1.06	4.54	1.00	0.765	0.995
25.0	4.16	8.73	25.1	6.04	1.85	1.84	11.0	0.758	7.79	2.84	1.03	5.16	1.00	1.36	1.97
20.4	3.98	13.0	18.9	4.27	1.78	1.58	7.71	0.577	6.74	2.57	1.06	4.43	1.00	0.842	0.787
17.5	4.67	14.0	17.2	3.95	1.83	1.65	7.12	0.543	4.92	1.94	0.980	3.40	1.00	0.524	0.556
15.9	4.60	17.1	14.8	3.30	1.78	1.51	5.94	0.480	4.66	1.87	1.00	3.22	1.00	0.438	0.364
17.5	5.03	8.42	12.5	3.62	1.56	1.56	6.58	0.673	4.15	1.68	0.899	3.10	1.00	0.633	0.725
12.7	4.75	16.1	7.79	2.05	1.45	1.20	3.70	0.403	3.36	1.44	0.950	2.49	1.00	0.300	0.173
11.5	4.91	9.07	5.00	1.76	1.22	1.15	3.19	0.439	2.13	1.02	0.795	1.84	1.00	0.271	0.168
9.20	4.71	14.8	3.49	1.14	1.14	0.942	2.07	0.336	1.84	0.922	0.827	1.59	1.00	0.167	0.0642
8.60	4.97	6.45	2.12	1.02	0.915	0.915	1.85	0.394	1.14	0.642	0.673	1.17	1.00	0.181	0.0772
6.25	4.64	12.9	1.26	0.547	0.831	0.692	1.01	0.271	0.901	0.541	0.702	0.930	1.00	0.0830	0.0197
5.00	4.61	11.7	0.671	0.348	0.677	0.570	0.650	0.239	0.597	0.398	0.638	0.686	1.00	0.0568	0.01000
4.75	4.77	6.13	0.462	0.319	0.575	0.553	0.592	0.250	0.444	0.317	0.564	0.565	1.00	0.0590	0.00995
3.85	4.54	10.4	0.307	0.198	0.522	0.448	0.381	0.204	0.374	0.281	0.576	0.485	1.00	0.0364	0.00457
3.75	4.83	4.30	0.200	0.187	0.426	0.432	0.351	0.219	0.289	0.230	0.513	0.411	1.00	0.0432	0.00496
2.85	4.48	8.82	0.114	0.0970	0.370	0.329	0.196	0.171	0.223	0.192	0.518	0.328	1.00	0.0216	0.00189

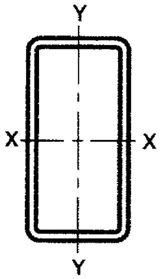
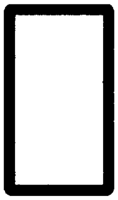


Table 1–11
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS20×12×5/8	0.581	127.00	35.0	17.7	31.4	1880	188	7.33	230
×1/2	0.465	103.00	28.3	22.8	40.0	1550	155	7.39	188
×3/8	0.349	78.45	21.5	31.4	54.3	1200	120	7.45	144
×5/16	0.291	65.82	18.1	38.2	65.7	1010	101	7.48	122
HSS20×8×5/8	0.581	110.00	30.3	10.8	31.4	1440	144	6.89	185
×1/2	0.465	89.55	24.6	14.2	40.0	1190	119	6.96	152
×3/8	0.349	68.29	18.7	19.9	54.3	926	92.6	7.03	117
×5/16	0.291	57.31	15.7	24.5	65.7	786	78.6	7.07	98.6
HSS20×4×1/2	0.465	75.94	20.9	5.60	40.0	838	83.8	6.33	115
×3/8	0.349	58.07	16.0	8.46	54.3	657	65.7	6.42	89.3
×5/16	0.291	48.87	13.4	10.7	65.7	560	56.0	6.46	75.6
×1/4	0.233	39.48	10.8	14.2	82.8	458	45.8	6.50	61.5
HSS18×6×5/8	0.581	93.10	25.7	7.33	28.0	923	103	6.00	135
×1/2	0.465	75.94	20.9	9.90	35.7	770	85.6	6.07	112
×3/8	0.349	58.07	16.0	14.2	48.6	602	66.9	6.15	86.4
×5/16	0.291	48.87	13.4	17.6	58.9	513	57.0	6.18	73.1
×1/4	0.233	39.48	10.8	22.8	74.3	419	46.5	6.22	59.4
HSS16×12×5/8	0.581	110.00	30.3	17.7	24.5	1090	136	6.00	165
×1/2	0.465	89.55	24.6	22.8	31.4	904	113	6.06	135
×3/8	0.349	68.29	18.7	31.4	42.8	702	87.7	6.12	104
×5/16	0.291	57.38	15.7	38.2	52.0	595	74.4	6.15	87.7
HSS16×8×5/8	0.581	93.10	25.7	10.8	24.5	815	102	5.64	129
×1/2	0.465	75.94	20.9	14.2	31.4	679	84.9	5.70	106
×3/8	0.349	58.07	16.0	19.9	42.8	531	66.3	5.77	82.1
×5/16	0.291	48.87	13.4	24.5	52.0	451	56.4	5.80	69.4
×1/4	0.233	39.48	10.8	31.3	65.7	368	46.1	5.83	56.4
HSS16×4×5/8	0.581	76.09	21.0	3.88	24.5	539	67.3	5.06	92.9
×1/2	0.465	62.33	17.2	5.60	31.4	455	56.9	5.15	77.3
×3/8	0.349	47.86	13.2	8.46	42.8	360	45.0	5.23	60.2
×5/16	0.291	40.35	11.1	10.7	52.0	308	38.5	5.27	51.1
×1/4	0.233	32.66	8.96	14.2	65.7	253	31.6	5.31	41.7
×3/16	0.174	24.66	6.76	20.0	89.0	193	24.2	5.35	31.7

Note: For compactness criteria, refer to the end of Table 1-12.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties



HSS20–HSS16

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS20×12×$^5/_8$	851	142	4.930	162	17$^3/_{16}$	9$^3/_{16}$	1890	257	5.17
×$^1/_2$	705	117	4.99	132	17$^3/_4$	9$^3/_4$	1540	209	5.20
×$^3/_8$	547	91.1	5.04	102	18$^5/_{16}$	10$^5/_{16}$	1180	160	5.23
×$^5/_{16}$	464	77.3	5.07	85.8	18$^5/_8$	10$^5/_8$	997	134	5.25
HSS20×8×$^5/_8$	338	84.6	3.34	96.4	17$^3/_{16}$	5$^3/_{16}$	916	167	4.50
×$^1/_2$	283	70.8	3.39	79.5	17$^3/_4$	5$^3/_4$	757	137	4.53
×$^3/_8$	222	55.6	3.44	61.5	18$^5/_{16}$	6$^5/_{16}$	586	105	4.57
×$^5/_{16}$	189	47.4	3.47	52.0	18$^5/_8$	6$^5/_8$	496	88.3	4.58
HSS20×4×$^1/_2$	58.7	29.3	1.68	34.0	17$^3/_4$	—	195	63.8	3.87
×$^3/_8$	47.6	23.8	1.73	26.8	18$^5/_{16}$	2$^5/_{16}$	156	49.9	3.90
×$^5/_{16}$	41.2	20.6	1.75	22.9	18$^5/_8$	2$^5/_8$	134	42.4	3.92
×$^1/_4$	34.3	17.1	1.78	18.7	18$^7/_8$	2$^7/_8$	111	34.7	3.93
HSS18×6×$^5/_8$	158	52.7	2.48	61.0	15$^3/_{16}$	3$^3/_{16}$	462	109	3.83
×$^1/_2$	134	44.6	2.53	50.7	15$^3/_4$	3$^3/_4$	387	89.9	3.87
×$^3/_8$	106	35.5	2.58	39.5	16$^5/_{16}$	4$^5/_{16}$	302	69.5	3.90
×$^5/_{16}$	91.3	30.4	2.61	33.5	16$^9/_{16}$	4$^9/_{16}$	257	58.7	3.92
×$^1/_4$	75.1	25.0	2.63	27.3	16$^7/_8$	4$^7/_8$	210	47.7	3.93
HSS16x12×$^5/_8$	700	117	4.80	135	13$^3/_{16}$	9$^3/_{16}$	1370	204	4.50
×$^1/_2$	581	96.8	4.86	111	13$^3/_4$	9$^3/_4$	1120	166	4.53
×$^3/_8$	452	75.3	4.91	85.5	14$^5/_{16}$	10$^5/_{16}$	862	127	4.57
×$^5/_{16}$	384	64.0	4.94	72.2	14$^5/_8$	10$^5/_8$	727	107	4.58
HSS16×8×$^5/_8$	274	68.6	3.27	79.2	13$^3/_{16}$	5$^3/_{16}$	681	132	3.83
×$^1/_2$	230	57.6	3.32	65.5	13$^3/_4$	5$^3/_4$	563	108	3.87
×$^3/_8$	181	45.3	3.37	50.8	14$^5/_{16}$	6$^5/_{16}$	436	83.4	3.90
×$^5/_{16}$	155	38.7	3.40	43.0	14$^5/_8$	6$^5/_8$	369	70.4	3.92
×$^1/_4$	127	31.7	3.42	35.0	14$^7/_8$	6$^7/_8$	300	57.0	3.93
HSS16×4×$^5/_8$	54.1	27.0	1.60	32.5	13$^3/_{16}$	—	174	60.5	3.17
×$^1/_2$	47.0	23.5	1.65	27.4	13$^3/_4$	—	150	50.7	3.20
×$^3/_8$	38.3	19.1	1.71	21.7	14$^5/_{16}$	2$^5/_{16}$	120	39.7	3.23
×$^5/_{16}$	33.2	16.6	1.73	18.5	14$^5/_8$	2$^5/_8$	103	33.8	3.25
×$^1/_4$	27.7	13.8	1.76	15.2	14$^7/_8$	2$^7/_8$	85.2	27.6	3.27
×$^3/_{16}$	21.5	10.8	1.78	11.7	15$^3/_{16}$	3$^3/_{16}$	65.5	21.1	3.28

—Flat depth or width is too small to establish a workable flat.

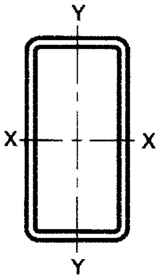


Table 1–11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS14×10×5/8	0.581	93.10	25.7	14.2	21.1	687	98.2	5.17	120
×1/2	0.465	75.94	20.9	18.5	27.1	573	81.8	5.23	98.8
×3/8	0.349	58.07	16.0	25.7	37.1	447	63.9	5.29	76.3
×5/16	0.291	48.87	13.4	31.4	45.1	380	54.3	5.32	64.6
×1/4	0.233	39.48	10.8	39.9	57.1	310	44.3	5.35	52.4
HSS14×6×5/8	0.581	76.09	21.0	7.33	21.1	478	68.3	4.77	88.7
×1/2	0.465	62.33	17.2	9.90	27.1	402	57.4	4.84	73.6
×3/8	0.349	47.86	13.2	14.2	37.1	317	45.3	4.91	57.3
×5/16	0.291	40.35	11.1	17.6	45.1	271	38.7	4.94	48.6
×1/4	0.233	32.66	8.96	22.8	57.1	222	31.7	4.98	39.6
×3/16	0.174	24.66	6.76	31.5	77.5	170	24.3	5.01	30.1
HSS14×4×5/8	0.581	67.59	18.7	3.88	21.1	373	53.3	4.47	73.1
×1/2	0.465	55.53	15.3	5.60	27.1	317	45.3	4.55	61.0
×3/8	0.349	42.75	11.8	8.46	37.1	252	36.0	4.63	47.8
×5/16	0.291	36.09	9.92	10.7	45.1	216	30.9	4.67	40.6
×1/4	0.233	29.25	8.03	14.2	57.1	178	25.4	4.71	33.2
×3/16	0.174	22.12	6.06	20.0	77.5	137	19.5	4.74	25.3
HSS12×10×1/2	0.465	69.14	19.0	18.5	22.8	395	65.9	4.56	78.8
×3/8	0.349	52.93	14.6	25.7	31.4	310	51.6	4.61	61.1
×5/16	0.291	44.62	12.2	31.4	38.2	264	44.0	4.64	51.7
×1/4	0.233	36.00	9.90	39.9	48.5	216	36.0	4.67	42.1
HSS12×8×5/8	0.581	76.13	21.0	10.8	17.7	397	66.1	4.34	82.1
×1/2	0.465	62.33	17.2	14.2	22.8	333	55.6	4.41	68.1
×3/8	0.349	47.82	13.2	19.9	31.4	262	43.7	4.47	53.0
×5/16	0.291	40.36	11.1	24.5	38.2	224	37.4	4.50	44.9
×1/4	0.233	32.60	8.96	31.3	48.5	184	30.6	4.53	36.6
×3/16	0.174	24.78	6.76	43.0	66.0	140	23.4	4.56	27.8
HSS12×6×5/8	0.581	67.62	18.7	7.33	17.7	321	53.4	4.14	68.8
×1/2	0.465	55.53	15.3	9.90	22.8	271	45.2	4.21	57.4
×3/8	0.349	42.72	11.8	14.2	31.4	215	35.9	4.28	44.8
×5/16	0.291	36.10	9.92	17.6	38.2	184	30.7	4.31	38.1
×1/4	0.233	29.19	8.03	22.8	48.5	151	25.2	4.34	31.1
×3/16	0.174	22.22	6.06	31.5	66.0	116	19.4	4.38	23.7

Note: For compactness criteria, refer to the end of Table 1-12.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS14–HSS12

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	S	r	Z	Depth	Width	J	C	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS14×10×$\frac{5}{8}$	407	81.5	3.98	95.1	$11\frac{3}{16}$	$7\frac{3}{16}$	832	146	3.83
×$\frac{1}{2}$	341	68.1	4.04	78.5	$11\frac{3}{4}$	$7\frac{3}{4}$	685	120	3.87
×$\frac{3}{8}$	267	53.4	4.09	60.7	$12\frac{5}{16}$	$8\frac{5}{16}$	528	91.8	3.90
×$\frac{5}{16}$	227	45.5	4.12	51.4	$12\frac{9}{16}$	$8\frac{9}{16}$	446	77.4	3.92
×$\frac{1}{4}$	186	37.2	4.14	41.8	$12\frac{7}{8}$	$8\frac{7}{8}$	362	62.6	3.93
HSS14×6×$\frac{5}{8}$	124	41.2	2.43	48.4	$11\frac{3}{16}$	$3\frac{3}{16}$	334	83.7	3.17
×$\frac{1}{2}$	105	35.1	2.48	40.4	$11\frac{3}{4}$	$3\frac{3}{4}$	279	69.3	3.20
×$\frac{3}{8}$	84.1	28.0	2.53	31.6	$12\frac{5}{16}$	$4\frac{5}{16}$	219	53.7	3.23
×$\frac{5}{16}$	72.3	24.1	2.55	26.9	$12\frac{9}{16}$	$4\frac{9}{16}$	186	45.5	3.25
×$\frac{1}{4}$	59.6	19.9	2.58	22.0	$12\frac{7}{8}$	$4\frac{7}{8}$	152	36.9	3.27
×$\frac{3}{16}$	45.9	15.3	2.61	16.7	$13\frac{3}{16}$	$5\frac{3}{16}$	116	28.0	3.28
HSS14×4×$\frac{5}{8}$	47.2	23.6	1.59	28.5	$11\frac{1}{4}$	—	148	52.6	2.83
×$\frac{1}{2}$	41.2	20.6	1.64	24.1	$11\frac{3}{4}$	—	127	44.1	2.87
×$\frac{3}{8}$	33.6	16.8	1.69	19.1	$12\frac{1}{4}$	$2\frac{1}{4}$	102	34.6	2.90
×$\frac{5}{16}$	29.2	14.6	1.72	16.4	$12\frac{5}{8}$	$2\frac{5}{8}$	87.7	29.5	2.92
×$\frac{1}{4}$	24.4	12.2	1.74	13.5	$12\frac{7}{8}$	$2\frac{7}{8}$	72.4	24.1	2.93
×$\frac{3}{16}$	19.0	9.48	1.77	10.3	$13\frac{1}{8}$	$3\frac{1}{8}$	55.8	18.4	2.95
HSS12×10×$\frac{1}{2}$	298	59.7	3.96	69.6	$9\frac{3}{4}$	$7\frac{3}{4}$	545	102	3.53
×$\frac{3}{8}$	234	46.9	4.01	54.0	$10\frac{5}{16}$	$8\frac{5}{16}$	421	78.3	3.57
×$\frac{5}{16}$	200	40.0	4.04	45.7	$10\frac{9}{16}$	$8\frac{9}{16}$	356	66.1	3.58
×$\frac{1}{4}$	164	32.7	4.07	37.2	$10\frac{7}{8}$	$8\frac{7}{8}$	289	53.5	3.60
HSS12×8×$\frac{5}{8}$	210	52.5	3.16	61.9	$9\frac{3}{16}$	$5\frac{3}{16}$	454	97.7	3.17
×$\frac{1}{2}$	178	44.4	3.21	51.5	$9\frac{3}{4}$	$5\frac{3}{4}$	377	80.4	3.20
×$\frac{3}{8}$	140	35.1	3.27	40.1	$10\frac{5}{16}$	$6\frac{5}{16}$	293	62.1	3.23
×$\frac{5}{16}$	120	30.1	3.29	34.1	$10\frac{9}{16}$	$6\frac{9}{16}$	248	52.4	3.25
×$\frac{1}{4}$	98.8	24.7	3.32	27.8	$10\frac{7}{8}$	$6\frac{7}{8}$	202	42.5	3.27
×$\frac{3}{16}$	75.7	18.9	3.35	21.1	$11\frac{1}{8}$	$7\frac{1}{8}$	153	32.2	3.28
HSS12×6×$\frac{5}{8}$	107	35.5	2.39	42.1	$9\frac{3}{16}$	$3\frac{3}{16}$	271	71.1	2.83
×$\frac{1}{2}$	91.1	30.4	2.44	35.2	$9\frac{3}{4}$	$3\frac{3}{4}$	227	59.0	2.87
×$\frac{3}{8}$	72.9	24.3	2.49	27.7	$10\frac{5}{16}$	$4\frac{5}{16}$	178	45.8	2.90
×$\frac{5}{16}$	62.8	20.9	2.52	23.6	$10\frac{9}{16}$	$4\frac{9}{16}$	152	38.8	2.92
×$\frac{1}{4}$	51.9	17.3	2.54	19.3	$10\frac{7}{8}$	$4\frac{7}{8}$	124	31.6	2.93
×$\frac{3}{16}$	40.0	13.3	2.57	14.7	$11\frac{3}{16}$	$5\frac{3}{16}$	94.6	24.0	2.95

—Flat depth or width is too small to establish a workable flat.

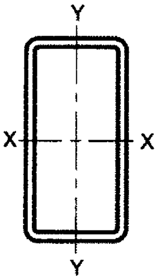
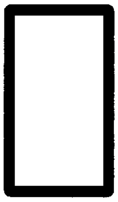


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS12×4×5/8	0.581	59.11	16.4	3.88	17.7	245	40.8	3.87	55.5
×1/2	0.465	48.72	13.5	5.60	22.8	210	34.9	3.95	46.7
×3/8	0.349	37.61	10.4	8.46	31.4	168	28.0	4.02	36.7
×5/16	0.291	31.84	8.76	10.7	38.2	144	24.1	4.06	31.3
×1/4	0.233	25.79	7.10	14.2	48.5	119	19.9	4.10	25.6
×3/16	0.174	19.66	5.37	20.0	66.0	91.8	15.3	4.13	19.6
HSS12×3½×3/8	0.349	36.34	10.0	7.03	31.4	156	26.0	3.94	34.7
×5/16	0.291	30.77	8.46	9.03	38.2	134	22.4	3.98	29.6
HSS12×3×5/16	0.291	29.71	8.17	7.31	38.2	124	20.7	3.90	27.9
×1/4	0.233	24.09	6.63	9.88	48.5	103	17.2	3.94	22.9
×3/16	0.174	18.38	5.02	14.2	66.0	79.6	13.3	3.98	17.5
HSS12×2×5/16	0.291	27.58	7.59	3.87	38.2	104	17.4	3.71	24.5
×1/4	0.233	22.39	6.17	5.58	48.5	86.9	14.5	3.75	20.1
×3/16	0.174	17.10	4.67	8.49	66.0	67.4	11.2	3.80	15.5
HSS10×8×5/8	0.581	67.62	18.7	10.8	14.2	253	50.5	3.68	62.2
×1/2	0.465	55.53	15.3	14.2	18.5	214	42.7	3.73	51.9
×3/8	0.349	42.72	11.8	19.9	25.7	169	33.9	3.79	40.5
×5/16	0.291	36.10	9.92	24.5	31.4	145	29.0	3.82	34.4
×1/4	0.233	29.19	8.03	31.3	39.9	119	23.8	3.85	28.1
×3/16	0.174	22.22	6.06	43.0	54.5	91.4	18.3	3.88	21.4
HSS10×6×5/8	0.581	59.11	16.4	7.33	14.2	201	40.2	3.50	51.3
×1/2	0.465	48.72	13.5	9.90	18.5	171	34.3	3.57	43.0
×3/8	0.349	37.61	10.4	14.2	25.7	137	27.4	3.63	33.8
×5/16	0.291	31.84	8.76	17.6	31.4	118	23.5	3.66	28.8
×1/4	0.233	25.79	7.10	22.8	39.9	96.9	19.4	3.69	23.6
×3/16	0.174	19.66	5.37	31.5	54.5	74.6	14.9	3.73	18.0
HSS10×5×3/8	0.349	35.06	9.67	11.3	25.7	120	24.1	3.53	30.4
×5/16	0.291	29.71	8.17	14.2	31.4	104	20.8	3.56	26.0
×1/4	0.233	24.09	6.63	18.5	39.9	85.8	17.2	3.60	21.3
×3/16	0.174	18.38	5.02	25.7	54.5	66.2	13.2	3.63	16.3

Note: For compactness criteria, refer to the end of Table 1-12.

Table 1–11 (continued)
Rectangular HSS
Dimensions and Properties



HSS12–HSS10

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS12×4×$\frac{5}{8}$	40.4	20.2	1.57	24.5	$9\frac{3}{16}$	—	122	44.6	2.50
×$\frac{1}{2}$	35.3	17.7	1.62	20.9	$9\frac{3}{4}$	—	105	37.5	2.53
×$\frac{3}{8}$	28.9	14.5	1.67	16.6	$10\frac{5}{16}$	$2\frac{5}{16}$	84.1	29.5	2.57
×$\frac{5}{16}$	25.2	12.6	1.70	14.2	$10\frac{5}{8}$	$2\frac{5}{8}$	72.4	25.2	2.58
×$\frac{1}{4}$	21.0	10.5	1.72	11.7	$10\frac{7}{8}$	$2\frac{7}{8}$	59.8	20.6	2.60
×$\frac{3}{16}$	16.4	8.20	1.75	9.00	$11\frac{3}{16}$	$3\frac{3}{16}$	46.1	15.7	2.62
HSS12×$3\frac{1}{2}$×$\frac{3}{8}$	21.3	12.2	1.46	14.0	$10\frac{5}{16}$	—	64.7	25.5	2.48
×$\frac{5}{16}$	18.6	10.6	1.48	12.1	$10\frac{5}{8}$	—	56.0	21.8	2.50
HSS12×3×$\frac{5}{16}$	13.1	8.73	1.27	10.0	$10\frac{5}{8}$	—	41.3	18.4	2.42
×$\frac{1}{4}$	11.1	7.38	1.29	8.28	$10\frac{7}{8}$	—	34.5	15.1	2.43
×$\frac{3}{16}$	8.72	5.81	1.32	6.40	$11\frac{3}{16}$	$2\frac{3}{16}$	26.8	11.6	2.45
HSS12×2×$\frac{5}{16}$	5.10	5.10	0.820	6.05	$10\frac{5}{8}$	—	17.6	11.6	2.25
×$\frac{1}{4}$	4.41	4.41	0.845	5.08	$10\frac{7}{8}$	—	15.1	9.64	2.27
×$\frac{3}{16}$	3.55	3.55	0.872	3.97	$11\frac{3}{16}$	—	12.0	7.49	2.28
HSS10×8×$\frac{5}{8}$	178	44.5	3.09	53.3	$7\frac{3}{16}$	$5\frac{3}{16}$	346	80.4	2.83
×$\frac{1}{2}$	151	37.8	3.14	44.5	$7\frac{3}{4}$	$5\frac{3}{4}$	288	66.4	2.87
×$\frac{3}{8}$	120	30.0	3.19	34.8	$8\frac{5}{16}$	$6\frac{5}{16}$	224	51.4	2.90
×$\frac{5}{16}$	103	25.7	3.22	29.6	$8\frac{5}{8}$	$6\frac{5}{8}$	190	43.5	2.92
×$\frac{1}{4}$	84.7	21.2	3.25	24.2	$8\frac{7}{8}$	$6\frac{7}{8}$	155	35.3	2.93
×$\frac{3}{16}$	65.1	16.3	3.28	18.4	$9\frac{3}{16}$	$7\frac{3}{16}$	118	26.7	2.95
HSS10×6×$\frac{5}{8}$	89.4	29.8	2.34	35.8	$7\frac{3}{16}$	$3\frac{3}{16}$	209	58.6	2.50
×$\frac{1}{2}$	76.8	25.6	2.39	30.1	$7\frac{3}{4}$	$3\frac{3}{4}$	176	48.7	2.53
×$\frac{3}{8}$	61.8	20.6	2.44	23.7	$8\frac{5}{16}$	$4\frac{5}{16}$	139	37.9	2.57
×$\frac{5}{16}$	53.3	17.8	2.47	20.2	$8\frac{5}{8}$	$4\frac{5}{8}$	118	32.2	2.58
×$\frac{1}{4}$	44.1	14.7	2.49	16.6	$8\frac{7}{8}$	$4\frac{7}{8}$	96.7	26.2	2.60
×$\frac{3}{16}$	34.1	11.4	2.52	12.7	$9\frac{3}{16}$	$5\frac{3}{16}$	73.8	19.9	2.62
HSS10×5×$\frac{3}{8}$	40.6	16.2	2.05	18.7	$8\frac{5}{16}$	$3\frac{5}{16}$	100	31.2	2.40
×$\frac{5}{16}$	35.2	14.1	2.07	16.0	$8\frac{5}{8}$	$3\frac{5}{8}$	86.0	26.5	2.42
×$\frac{1}{4}$	29.3	11.7	2.10	13.2	$8\frac{7}{8}$	$3\frac{7}{8}$	70.7	21.6	2.43
×$\frac{3}{16}$	22.7	9.09	2.13	10.1	$9\frac{3}{16}$	$4\frac{3}{16}$	54.1	16.5	2.45

—Flat depth or width is too small to establish a workable flat.

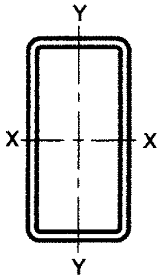


Table 1-11 (continued)

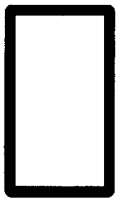
Rectangular HSS

Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS10×4×5/8	0.581	50.60	14.0	3.88	14.2	149	29.9	3.26	40.3
×1/2	0.465	41.91	11.6	5.60	18.5	129	25.8	3.34	34.1
×3/8	0.349	32.51	8.97	8.46	25.7	104	20.8	3.41	27.0
×5/16	0.291	27.58	7.59	10.7	31.4	90.1	18.0	3.44	23.1
×1/4	0.233	22.39	6.17	14.2	39.9	74.7	14.9	3.48	19.0
×3/16	0.174	17.10	4.67	20.0	54.5	57.8	11.6	3.52	14.6
×1/8	0.116	11.55	3.16	31.5	83.2	39.8	7.97	3.55	9.95
HSS10×3-1/2×1/2	0.465	40.21	11.1	4.53	18.5	118	23.7	3.26	31.9
×3/8	0.349	31.23	8.62	7.03	25.7	96.1	19.2	3.34	25.3
×5/16	0.291	26.51	7.30	9.03	31.4	83.2	16.6	3.38	21.7
×1/4	0.233	21.54	5.93	12.0	39.9	69.1	13.8	3.41	17.9
×3/16	0.174	16.46	4.50	17.1	54.5	53.6	10.7	3.45	13.7
×1/8	0.116	11.13	3.04	27.2	83.2	37.0	7.40	3.49	9.37
HSS10×3×3/8	0.349	29.96	8.27	5.60	25.7	88.0	17.6	3.26	23.7
×5/16	0.291	25.45	7.01	7.31	31.4	76.3	15.3	3.30	20.3
×1/4	0.233	20.69	5.70	9.88	39.9	63.6	12.7	3.34	16.7
×3/16	0.174	15.82	4.32	14.2	54.5	49.4	9.87	3.38	12.8
×1/8	0.116	10.70	2.93	22.9	83.2	34.2	6.83	3.42	8.80
HSS10×2×3/8	0.349	27.41	7.58	2.73	25.7	71.7	14.3	3.08	20.3
×5/16	0.291	23.32	6.43	3.87	31.4	62.6	12.5	3.12	17.5
×1/4	0.233	18.99	5.24	5.58	39.9	52.5	10.5	3.17	14.4
×3/16	0.174	14.54	3.98	8.49	54.5	41.0	8.19	3.21	11.1
×1/8	0.116	9.85	2.70	14.2	83.2	28.5	5.70	3.25	7.65
HSS9×7×5/8	0.581	59.11	16.4	9.05	12.5	174	38.7	3.26	48.3
×1/2	0.465	48.72	13.5	12.1	16.4	149	33.0	3.32	40.5
×3/8	0.349	37.61	10.4	17.1	22.8	119	26.4	3.38	31.8
×5/16	0.291	31.84	8.76	21.1	27.9	102	22.6	3.41	27.1
×1/4	0.233	25.79	7.10	27.0	35.6	84.1	18.7	3.44	22.2
×3/16	0.174	19.66	5.37	37.2	48.7	64.7	14.4	3.47	16.9

Note: For compactness criteria, refer to the end of Table 1-12.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties



HSS10–HSS9

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS10×4×$^5/_8$	33.5	16.8	1.54	20.6	$7^3/_{16}$	—	95.7	36.7	2.17
×$^1/_2$	29.5	14.7	1.59	17.6	$7^3/_4$	—	82.6	31.0	2.20
×$^3/_8$	24.3	12.1	1.64	14.0	$8^5/_{16}$	$2^5/_{16}$	66.5	24.4	2.23
×$^5/_{16}$	21.2	10.6	1.67	12.1	$8^5/_8$	$2^5/_8$	57.3	20.9	2.25
×$^1/_4$	17.7	8.87	1.70	10.0	$8^7/_8$	$2^7/_8$	47.4	17.1	2.27
×$^3/_{16}$	13.9	6.93	1.72	7.66	$9^3/_{16}$	$3^3/_{16}$	36.5	13.1	2.28
×$^1/_8$	9.65	4.83	1.75	5.26	$9^7/_{16}$	$3^7/_{16}$	25.1	8.90	2.30
HSS10×3$^1/_2$×$^1/_2$	21.4	12.2	1.39	14.7	$7^3/_4$	—	63.2	26.5	2.12
×$^3/_8$	17.8	10.2	1.44	11.8	$8^5/_{16}$	—	51.5	21.1	2.15
×$^5/_{16}$	15.6	8.92	1.46	10.2	$8^5/_8$	—	44.6	18.0	2.17
×$^1/_4$	13.1	7.51	1.49	8.45	$8^7/_8$	—	37.0	14.8	2.18
×$^3/_{16}$	10.3	5.89	1.51	6.52	$9^3/_{16}$	$2^{11}/_{16}$	28.6	11.4	2.20
×$^1/_8$	7.22	4.12	1.54	4.48	$9^7/_{16}$	$2^{15}/_{16}$	19.8	7.75	2.22
HSS10×3×$^3/_8$	12.4	8.28	1.22	9.73	$8^5/_{16}$	—	37.8	17.7	2.07
×$^5/_{16}$	11.0	7.30	1.25	8.42	$8^5/_8$	—	33.0	15.2	2.08
×$^1/_4$	9.28	6.19	1.28	6.99	$8^7/_8$	—	27.6	12.5	2.10
×$^3/_{16}$	7.33	4.89	1.30	5.41	$9^3/_{16}$	$2^3/_{16}$	21.5	9.64	2.12
×$^1/_8$	5.16	3.44	1.33	3.74	$9^7/_{16}$	$2^7/_{16}$	14.9	6.61	2.13
HSS10×2×$^3/_8$	4.70	4.70	0.787	5.76	$8^5/_{16}$	—	15.9	11.0	1.90
×$^5/_{16}$	4.24	4.24	0.812	5.06	$8^5/_8$	—	14.2	9.56	1.92
×$^1/_4$	3.67	3.67	0.838	4.26	$8^7/_8$	—	12.2	7.99	1.93
×$^3/_{16}$	2.97	2.97	0.864	3.34	$9^3/_{16}$	—	9.74	6.22	1.95
×$^1/_8$	2.14	2.14	0.890	2.33	$9^7/_{16}$	—	6.90	4.31	1.97
HSS9×7×$^5/_8$	117	33.5	2.68	40.5	$6^3/_{16}$	$4^3/_{16}$	235	62.0	2.50
×$^1/_2$	100	28.7	2.73	34.0	$6^3/_4$	$4^3/_4$	197	51.5	2.53
×$^3/_8$	80.4	23.0	2.78	26.7	$7^5/_{16}$	$5^5/_{16}$	154	40.0	2.57
×$^5/_{16}$	69.2	19.8	2.81	22.8	$7^5/_8$	$5^5/_8$	131	33.9	2.58
×$^1/_4$	57.2	16.3	2.84	18.7	$7^7/_8$	$5^7/_8$	107	27.6	2.60
×$^3/_{16}$	44.1	12.6	2.87	14.3	$8^3/_{16}$	$6^3/_{16}$	81.7	20.9	2.62

—Flat depth or width is too small to establish a workable flat.

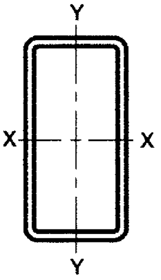
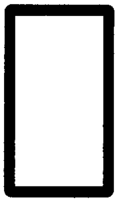


Table 1–11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS9×5×5/8	0.581	50.60	14.0	5.61	12.5	133	29.6	3.08	38.5
×1/2	0.465	41.91	11.6	7.75	16.4	115	25.5	3.14	32.5
×3/8	0.349	32.51	8.97	11.3	22.8	92.5	20.5	3.21	25.7
×5/16	0.291	27.58	7.59	14.2	27.9	79.8	17.7	3.24	22.0
×1/4	0.233	22.39	6.17	18.5	35.6	66.1	14.7	3.27	18.1
×3/16	0.174	17.10	4.67	25.7	48.7	51.1	11.4	3.31	13.8
HSS9×3×1/2	0.465	35.11	9.74	3.45	16.4	80.8	18.0	2.88	24.6
×3/8	0.349	27.41	7.58	5.60	22.8	66.3	14.7	2.96	19.7
×5/16	0.291	23.32	6.43	7.31	27.9	57.7	12.8	3.00	16.9
×1/4	0.233	18.99	5.24	9.88	35.6	48.2	10.7	3.04	14.0
×3/16	0.174	14.54	3.98	14.2	48.7	37.6	8.35	3.07	10.8
HSS8×6×5/8	0.581	50.60	14.0	7.33	10.8	114	28.5	2.85	36.1
×1/2	0.465	41.91	11.6	9.90	14.2	98.2	24.6	2.91	30.5
×3/8	0.349	32.51	8.97	14.2	19.9	79.1	19.8	2.97	24.1
×5/16	0.291	27.58	7.59	17.6	24.5	68.3	17.1	3.00	20.6
×1/4	0.233	22.39	6.17	22.8	31.3	56.6	14.2	3.03	16.9
×3/16	0.174	17.10	4.67	31.5	43.0	43.7	10.9	3.06	13.0
HSS8×4×5/8	0.581	42.10	11.7	3.88	10.8	82.0	20.5	2.64	27.4
×1/2	0.465	35.11	9.74	5.60	14.2	71.8	17.9	2.71	23.5
×3/8	0.349	27.41	7.58	8.46	19.9	58.7	14.7	2.78	18.8
×5/16	0.291	23.32	6.43	10.7	24.5	51.0	12.8	2.82	16.1
×1/4	0.233	18.99	5.24	14.2	31.3	42.5	10.6	2.85	13.3
×3/16	0.174	14.54	3.98	20.0	43.0	33.1	8.27	2.88	10.2
×1/8	0.116	9.85	2.70	31.5	66.0	22.9	5.73	2.92	7.02
HSS8×3×1/2	0.465	31.71	8.81	3.45	14.2	58.6	14.6	2.58	20.0
×3/8	0.349	24.85	6.88	5.60	19.9	48.5	12.1	2.65	16.1
×5/16	0.291	21.19	5.85	7.31	24.5	42.4	10.6	2.69	13.9
×1/4	0.233	17.28	4.77	9.88	31.3	35.5	8.88	2.73	11.5
×3/16	0.174	13.26	3.63	14.2	43.0	27.8	6.94	2.77	8.87
×1/8	0.116	9.00	2.46	22.9	66.0	19.3	4.83	2.80	6.11

Note: For compactness criteria, refer to the end of Table 1-12.

Table 1–11 (continued)
Rectangular HSS
Dimensions and Properties



HSS9–HSS8

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS9×5×$\frac{5}{8}$	52.0	20.8	1.92	25.3	$6\frac{3}{16}$	$2\frac{3}{16}$	128	42.5	2.17
×$\frac{1}{2}$	45.2	18.1	1.97	21.5	$6\frac{3}{4}$	$2\frac{3}{4}$	109	35.6	2.20
×$\frac{3}{8}$	36.8	14.7	2.03	17.1	$7\frac{5}{16}$	$3\frac{5}{16}$	86.9	27.9	2.23
×$\frac{5}{16}$	32.0	12.8	2.05	14.6	$7\frac{5}{8}$	$3\frac{5}{8}$	74.4	23.8	2.25
×$\frac{1}{4}$	26.6	10.6	2.08	12.0	$7\frac{7}{8}$	$3\frac{7}{8}$	61.2	19.4	2.27
×$\frac{3}{16}$	20.7	8.28	2.10	9.25	$8\frac{3}{16}$	$4\frac{3}{16}$	46.9	14.8	2.28
HSS9×3×$\frac{1}{2}$	13.2	8.81	1.17	10.8	$6\frac{3}{4}$	—	40.0	19.7	1.87
×$\frac{3}{8}$	11.2	7.45	1.21	8.80	$7\frac{5}{16}$	—	33.1	15.8	1.90
×$\frac{5}{16}$	9.88	6.59	1.24	7.63	$7\frac{5}{8}$	—	28.9	13.6	1.92
×$\frac{1}{4}$	8.38	5.59	1.27	6.35	$7\frac{7}{8}$	—	24.2	11.3	1.93
×$\frac{3}{16}$	6.64	4.42	1.29	4.92	$8\frac{3}{16}$	$2\frac{3}{16}$	18.9	8.66	1.95
HSS8×6×$\frac{5}{8}$	72.3	24.1	2.27	29.5	$5\frac{3}{16}$	$3\frac{3}{16}$	150	46.0	2.17
×$\frac{1}{2}$	62.5	20.8	2.32	24.9	$5\frac{3}{4}$	$3\frac{3}{4}$	127	38.4	2.20
×$\frac{3}{8}$	50.6	16.9	2.38	19.8	$6\frac{5}{16}$	$4\frac{5}{16}$	100	30.0	2.23
×$\frac{5}{16}$	43.8	14.6	2.40	16.9	$6\frac{5}{8}$	$4\frac{5}{8}$	85.8	25.5	2.25
×$\frac{1}{4}$	36.4	12.1	2.43	13.9	$6\frac{7}{8}$	$4\frac{7}{8}$	70.3	20.8	2.27
×$\frac{3}{16}$	28.2	9.39	2.46	10.7	$7\frac{3}{16}$	$5\frac{3}{16}$	53.7	15.8	2.28
HSS8×4×$\frac{5}{8}$	26.6	13.3	1.51	16.6	$5\frac{3}{16}$	—	70.3	28.7	1.83
×$\frac{1}{2}$	23.6	11.8	1.56	14.3	$5\frac{3}{4}$	—	61.1	24.4	1.87
×$\frac{3}{8}$	19.6	9.80	1.61	11.5	$6\frac{5}{16}$	$2\frac{5}{16}$	49.3	19.3	1.90
×$\frac{5}{16}$	17.2	8.58	1.63	9.91	$6\frac{5}{8}$	$2\frac{5}{8}$	42.6	16.5	1.92
×$\frac{1}{4}$	14.4	7.21	1.66	8.20	$6\frac{7}{8}$	$2\frac{7}{8}$	35.3	13.6	1.93
×$\frac{3}{16}$	11.3	5.65	1.69	6.33	$7\frac{3}{16}$	$3\frac{3}{16}$	27.2	10.4	1.95
×$\frac{1}{8}$	7.90	3.95	1.71	4.36	$7\frac{7}{16}$	$3\frac{7}{16}$	18.7	7.10	1.97
HSS8×3×$\frac{1}{2}$	11.7	7.81	1.15	9.64	$5\frac{3}{4}$	—	34.3	17.4	1.70
×$\frac{3}{8}$	9.95	6.63	1.20	7.88	$6\frac{5}{16}$	—	28.5	14.0	1.73
×$\frac{5}{16}$	8.81	5.87	1.23	6.84	$6\frac{5}{8}$	—	24.9	12.1	1.75
×$\frac{1}{4}$	7.49	4.99	1.25	5.70	$6\frac{7}{8}$	—	20.8	10.0	1.77
×$\frac{3}{16}$	5.94	3.96	1.28	4.43	$7\frac{3}{16}$	$2\frac{3}{16}$	16.2	7.68	1.78
×$\frac{1}{8}$	4.20	2.80	1.31	3.07	$7\frac{7}{16}$	$2\frac{7}{16}$	11.3	5.27	1.80

—Flat depth or width is too small to establish a workable flat.

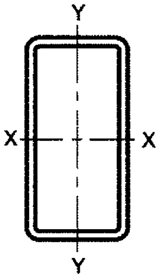


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS8×2×3/8	0.349	22.30	6.18	2.73	19.9	38.2	9.56	2.49	13.4
×5/16	0.291	19.06	5.26	3.87	24.5	33.7	8.43	2.53	11.6
×1/4	0.233	15.58	4.30	5.58	31.3	28.5	7.12	2.57	9.68
×3/16	0.174	11.98	3.28	8.49	43.0	22.4	5.61	2.61	7.51
×1/8	0.116	8.15	2.23	14.2	66.0	15.7	3.93	2.65	5.19
HSS7×5×1/2	0.465	35.11	9.74	7.75	12.1	60.6	17.3	2.50	21.9
×3/8	0.349	27.41	7.58	11.3	17.1	49.5	14.1	2.56	17.5
×5/16	0.291	23.32	6.43	14.2	21.1	43.0	12.3	2.59	15.0
×1/4	0.233	18.99	5.24	18.5	27.0	35.9	10.2	2.62	12.4
×3/16	0.174	14.54	3.98	25.7	37.2	27.9	7.96	2.65	9.52
×1/8	0.116	9.85	2.70	40.1	57.3	19.3	5.52	2.68	6.53
HSS7×4×1/2	0.465	31.71	8.81	5.60	12.1	50.7	14.5	2.40	18.8
×3/8	0.349	24.85	6.88	8.46	17.1	41.8	11.9	2.46	15.1
×5/16	0.291	21.19	5.85	10.7	21.1	36.5	10.4	2.50	13.1
×1/4	0.233	17.28	4.77	14.2	27.0	30.5	8.72	2.53	10.8
×3/16	0.174	13.26	3.63	20.0	37.2	23.8	6.81	2.56	8.33
×1/8	0.116	9.00	2.46	31.5	57.3	16.6	4.73	2.59	5.73
HSS7×3×1/2	0.465	28.30	7.88	3.45	12.1	40.7	11.6	2.27	15.8
×3/8	0.349	22.30	6.18	5.60	17.1	34.1	9.73	2.35	12.8
×5/16	0.291	19.06	5.26	7.31	21.1	29.9	8.54	2.38	11.1
×1/4	0.233	15.58	4.30	9.88	27.0	25.2	7.19	2.42	9.22
×3/16	0.174	11.98	3.28	14.2	37.2	19.8	5.65	2.45	7.14
×1/8	0.116	8.15	2.23	22.9	57.3	13.8	3.95	2.49	4.93
HSS7×2×1/4	0.233	13.88	3.84	5.58	27.0	19.8	5.67	2.27	7.64
×3/16	0.174	10.70	2.93	8.49	37.2	15.7	4.49	2.31	5.95
×1/8	0.116	7.30	2.00	14.2	57.3	11.1	3.16	2.35	4.13
HSS6×5×1/2	0.465	31.71	8.81	7.75	9.90	41.1	13.7	2.16	17.2
×3/8	0.349	24.85	6.88	11.3	14.2	33.9	11.3	2.22	13.8
×5/16	0.291	21.19	5.85	14.2	17.6	29.6	9.85	2.25	11.9
×1/4	0.233	17.28	4.77	18.5	22.8	24.7	8.25	2.28	9.87
×3/16	0.174	13.26	3.63	25.7	31.5	19.3	6.44	2.31	7.62
×1/8	0.116	9.00	2.46	40.1	48.7	13.4	4.48	2.34	5.24

Note: For compactness criteria, refer to the end of Table 1-12.

Table 1–11 (continued)
Rectangular HSS
Dimensions and Properties

HSS8–HSS6

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS8×2×3/8	3.73	3.73	0.777	4.61	6 5/16	—	12.1	8.65	1.57
×5/16	3.38	3.38	0.802	4.06	6 5/8	—	10.9	7.57	1.58
×1/4	2.94	2.94	0.827	3.43	6 7/8	—	9.36	6.35	1.60
×3/16	2.39	2.39	0.853	2.70	7 3/16	—	7.48	4.95	1.62
×1/8	1.72	1.72	0.879	1.90	7 7/16	—	5.30	3.44	1.63
HSS7×5×1/2	35.6	14.2	1.91	17.3	4 3/4	2 3/4	75.8	27.2	1.87
×3/8	29.3	11.7	1.97	13.8	5 5/16	3 5/16	60.6	21.4	1.90
×5/16	25.5	10.2	1.99	11.9	5 5/8	3 5/8	52.1	18.3	1.92
×1/4	21.3	8.53	2.02	9.83	5 7/8	3 7/8	42.9	15.0	1.93
×3/16	16.6	6.65	2.05	7.57	6 3/16	4 3/16	32.9	11.4	1.95
×1/8	11.6	4.63	2.07	5.20	6 7/16	4 7/16	22.5	7.79	1.97
HSS7×4×1/2	20.7	10.4	1.53	12.6	4 3/4	—	50.5	21.1	1.70
×3/8	17.3	8.63	1.58	10.2	5 5/16	2 5/16	41.0	16.8	1.73
×5/16	15.2	7.58	1.61	8.83	5 5/8	2 5/8	35.4	14.4	1.75
×1/4	12.8	6.38	1.64	7.33	5 7/8	2 7/8	29.3	11.8	1.77
×3/16	10.0	5.02	1.66	5.67	6 1/8	3 1/8	22.7	9.07	1.78
×1/8	7.03	3.51	1.69	3.91	6 7/16	3 7/16	15.6	6.20	1.80
HSS7×3×1/2	10.2	6.80	1.14	8.46	4 3/4	—	28.6	15.0	1.53
×3/8	8.71	5.81	1.19	6.95	5 5/16	—	23.9	12.1	1.57
×5/16	7.74	5.16	1.21	6.05	5 5/8	—	20.9	10.5	1.58
×1/4	6.60	4.40	1.24	5.06	5 7/8	—	17.5	8.68	1.60
×3/16	5.24	3.50	1.26	3.94	6 3/16	2 3/16	13.7	6.69	1.62
×1/8	3.71	2.48	1.29	2.73	6 7/16	2 7/16	9.48	4.60	1.63
HSS7×2×1/4	2.58	2.58	0.819	3.02	5 7/8	—	7.95	5.52	1.43
×3/16	2.10	2.10	0.845	2.39	6 3/16	—	6.35	4.32	1.45
×1/8	1.52	1.52	0.871	1.68	6 7/16	—	4.51	3.00	1.47
HSS6×5×1/2	30.8	12.3	1.87	15.2	3 3/4	2 3/4	59.8	23.0	1.70
×3/8	25.5	10.2	1.92	12.2	4 5/16	3 5/16	48.1	18.2	1.73
×5/16	22.3	8.91	1.95	10.5	4 5/8	3 5/8	41.4	15.6	1.75
×1/4	18.7	7.47	1.98	8.72	4 7/8	3 7/8	34.2	12.8	1.77
×3/16	14.6	5.84	2.01	6.73	5 3/16	4 3/16	26.3	9.76	1.78
×1/8	10.2	4.07	2.03	4.63	5 7/16	4 7/16	18.0	6.66	1.80

—Flat depth or width is too small to establish a workable flat.

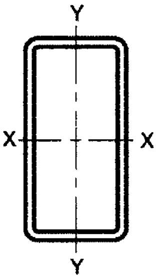


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS6×4×1/2	0.465	28.30	7.88	5.60	9.90	34.0	11.3	2.08	14.6
×3/8	0.349	22.30	6.18	8.46	14.2	28.3	9.43	2.14	11.9
×5/16	0.291	19.06	5.26	10.7	17.6	24.8	8.27	2.17	10.3
×1/4	0.233	15.58	4.30	14.2	22.8	20.9	6.96	2.20	8.53
×3/16	0.174	11.98	3.28	20.0	31.5	16.4	5.46	2.23	6.60
×1/8	0.116	8.15	2.23	31.5	48.7	11.4	3.81	2.26	4.56
HSS6×3×1/2	0.465	24.90	6.95	3.45	9.90	26.8	8.95	1.97	12.1
×3/8	0.349	19.75	5.48	5.60	14.2	22.7	7.57	2.04	9.90
×5/16	0.291	16.93	4.68	7.31	17.6	20.1	6.69	2.07	8.61
×1/4	0.233	13.88	3.84	9.88	22.8	17.0	5.66	2.10	7.19
×3/16	0.174	10.70	2.93	14.2	31.5	13.4	4.47	2.14	5.59
×1/8	0.116	7.30	2.00	22.9	48.7	9.43	3.14	2.17	3.87
HSS6×2×3/8	0.349	17.20	4.78	2.73	14.2	17.1	5.71	1.89	7.93
×5/16	0.291	14.80	4.10	3.87	17.6	15.3	5.11	1.93	6.95
×1/4	0.233	12.18	3.37	5.58	22.8	13.1	4.37	1.97	5.84
×3/16	0.174	9.43	2.58	8.49	31.5	10.5	3.49	2.01	4.58
×1/8	0.116	6.45	1.77	14.2	48.7	7.42	2.47	2.05	3.19
HSS5×4×1/2	0.465	24.90	6.95	5.60	7.75	21.2	8.49	1.75	10.9
×3/8	0.349	19.75	5.48	8.46	11.3	17.9	7.17	1.81	8.96
×5/16	0.291	16.93	4.68	10.7	14.2	15.8	6.32	1.84	7.79
×1/4	0.233	13.88	3.84	14.2	18.5	13.4	5.35	1.87	6.49
×3/16	0.174	10.70	2.93	20.0	25.7	10.6	4.22	1.90	5.05
×1/8	0.116	7.30	2.00	31.5	40.1	7.42	2.97	1.93	3.50
HSS5×3×1/2	0.465	21.50	6.02	3.45	7.75	16.4	6.57	1.65	8.83
×3/8	0.349	17.20	4.78	5.60	11.3	14.1	5.65	1.72	7.34
×5/16	0.291	14.80	4.10	7.31	14.2	12.6	5.03	1.75	6.42
×1/4	0.233	12.18	3.37	9.88	18.5	10.7	4.29	1.78	5.38
×3/16	0.174	9.43	2.58	14.2	25.7	8.53	3.41	1.82	4.21
×1/8	0.116	6.45	1.77	22.9	40.1	6.03	2.41	1.85	2.93

Note: For compactness criteria, refer to the end of Table 1-12.

Table 1-11 (continued)

Rectangular HSS

Dimensions and Properties

HSS6–HSS5

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS6×4×1/2	17.8	8.89	1.50	11.0	3 3/4	—	40.3	17.8	1.53
×3/8	14.9	7.47	1.55	8.94	4 5/16	2 5/16	32.8	14.2	1.57
×5/16	13.2	6.58	1.58	7.75	4 5/8	2 5/8	28.4	12.2	1.58
×1/4	11.1	5.56	1.61	6.45	4 7/8	2 7/8	23.6	10.1	1.60
×3/16	8.76	4.38	1.63	5.00	5 3/16	3 3/16	18.2	7.74	1.62
×1/8	6.15	3.08	1.66	3.46	5 7/16	3 7/16	12.6	5.30	1.63
HSS6×3×1/2	8.69	5.79	1.12	7.28	3 3/4	—	23.1	12.7	1.37
×3/8	7.48	4.99	1.17	6.03	4 5/16	—	19.3	10.3	1.40
×5/16	6.67	4.45	1.19	5.27	4 5/8	—	16.9	8.91	1.42
×1/4	5.70	3.80	1.22	4.41	4 7/8	—	14.2	7.39	1.43
×3/16	4.55	3.03	1.25	3.45	5 3/16	2 3/16	11.1	5.71	1.45
×1/8	3.23	2.15	1.27	2.40	5 7/16	2 7/16	7.73	3.93	1.47
HSS6×2×3/8	2.77	2.77	0.760	3.46	4 5/16	—	8.42	6.35	1.23
×5/16	2.52	2.52	0.785	3.07	4 5/8	—	7.60	5.58	1.25
×1/4	2.21	2.21	0.810	2.61	4 7/8	—	6.55	4.70	1.27
×3/16	1.80	1.80	0.836	2.07	5 3/16	—	5.24	3.68	1.28
×1/8	1.31	1.31	0.861	1.46	5 7/16	—	3.72	2.57	1.30
HSS5×4×1/2	14.9	7.43	1.46	9.35	2 3/4	—	30.3	14.5	1.37
×3/8	12.6	6.30	1.52	7.67	3 5/16	2 5/16	24.9	11.7	1.40
×5/16	11.1	5.57	1.54	6.67	3 5/8	2 5/8	21.7	10.1	1.42
×1/4	9.46	4.73	1.57	5.57	3 7/8	2 7/8	18.0	8.32	1.43
×3/16	7.48	3.74	1.60	4.34	4 3/16	3 3/16	14.0	6.41	1.45
×1/8	5.27	2.64	1.62	3.01	4 7/16	3 7/16	9.66	4.39	1.47
HSS5×3×1/2	7.18	4.78	1.09	6.10	2 3/4	—	17.6	10.3	1.20
×3/8	6.25	4.16	1.14	5.10	3 5/16	—	14.9	8.44	1.23
×5/16	5.60	3.73	1.17	4.48	3 5/8	—	13.1	7.33	1.25
×1/4	4.81	3.21	1.19	3.77	3 7/8	—	11.0	6.10	1.27
×3/16	3.85	2.57	1.22	2.96	4 3/16	2 3/16	8.64	4.73	1.28
×1/8	2.75	1.83	1.25	2.07	4 7/16	2 7/16	6.02	3.26	1.30

—Flat depth or width is too small to establish a workable flat.

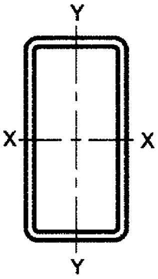


Table 1–11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS5×2½×¼	0.233	11.33	3.14	7.73	18.5	9.40	3.76	1.73	4.83
×3/16	0.174	8.79	2.41	11.4	25.7	7.51	3.01	1.77	3.79
×⅛	0.116	6.02	1.65	18.6	40.1	5.34	2.14	1.80	2.65
HSS5×2×⅜	0.349	14.65	4.09	2.73	11.3	10.4	4.14	1.59	5.71
×5/16	0.291	12.67	3.52	3.87	14.2	9.35	3.74	1.63	5.05
×¼	0.233	10.48	2.91	5.58	18.5	8.08	3.23	1.67	4.27
×3/16	0.174	8.15	2.24	8.49	25.7	6.50	2.60	1.70	3.37
×⅛	0.116	5.60	1.54	14.2	40.1	4.65	1.86	1.74	2.37
HSS4×3×⅜	0.349	14.65	4.09	5.60	8.46	7.93	3.97	1.39	5.12
×5/16	0.291	12.67	3.52	7.31	10.7	7.14	3.57	1.42	4.51
×¼	0.233	10.48	2.91	9.88	14.2	6.15	3.07	1.45	3.81
×3/16	0.174	8.15	2.24	14.2	20.0	4.93	2.47	1.49	3.00
×⅛	0.116	5.60	1.54	22.9	31.5	3.52	1.76	1.52	2.11
HSS4×2½×⅜	0.349	13.37	3.74	4.16	8.46	6.77	3.38	1.35	4.48
×5/16	0.291	11.60	3.23	5.59	10.7	6.13	3.07	1.38	3.97
×¼	0.233	9.63	2.67	7.73	14.2	5.32	2.66	1.41	3.38
×3/16	0.174	7.51	2.06	11.4	20.0	4.30	2.15	1.44	2.67
×⅛	0.116	5.17	1.42	18.6	31.5	3.09	1.54	1.47	1.88
HSS4×2×⅜	0.349	12.09	3.39	2.73	8.46	5.60	2.80	1.29	3.84
×5/16	0.291	10.54	2.94	3.87	10.7	5.13	2.56	1.32	3.43
×¼	0.233	8.78	2.44	5.58	14.2	4.49	2.25	1.36	2.94
×3/16	0.174	6.87	1.89	8.49	20.0	3.66	1.83	1.39	2.34
×⅛	0.116	4.75	1.30	14.2	31.5	2.65	1.32	1.43	1.66
HSS3½×2½×⅜	0.349	12.09	3.39	4.16	7.03	4.75	2.72	1.18	3.59
×5/16	0.291	10.54	2.94	5.59	9.03	4.34	2.48	1.22	3.20
×¼	0.233	8.78	2.44	7.73	12.0	3.79	2.17	1.25	2.74
×3/16	0.174	6.87	1.89	11.4	17.1	3.09	1.76	1.28	2.18
×⅛	0.116	4.75	1.30	18.6	27.2	2.23	1.28	1.31	1.54
HSS3½×2×¼	0.233	7.93	2.21	5.58	12.0	3.17	1.81	1.20	2.36
×3/16	0.174	6.23	1.71	8.49	17.1	2.61	1.49	1.23	1.89
×⅛	0.116	4.32	1.19	14.2	27.2	1.90	1.09	1.27	1.34

Note: For compactness criteria, refer to the end of Table 1-12.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS5–HSS3$\frac{1}{2}$

Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS5×2$\frac{1}{2}$×$\frac{1}{4}$	3.13	2.50	0.999	2.95	$3\frac{7}{8}$	—	7.93	4.99	1.18
×$\frac{3}{16}$	2.53	2.03	1.02	2.33	$4\frac{3}{16}$	—	6.26	3.89	1.20
×$\frac{1}{8}$	1.82	1.46	1.05	1.64	$4\frac{7}{16}$	—	4.40	2.70	1.22
HSS5×2×$\frac{3}{8}$	2.28	2.28	0.748	2.88	$3\frac{5}{16}$	—	6.61	5.20	1.07
×$\frac{5}{16}$	2.10	2.10	0.772	2.57	$3\frac{5}{8}$	—	5.99	4.59	1.08
×$\frac{1}{4}$	1.84	1.84	0.797	2.20	$3\frac{7}{8}$	—	5.17	3.88	1.10
×$\frac{3}{16}$	1.51	1.51	0.823	1.75	$4\frac{3}{16}$	—	4.15	3.05	1.12
×$\frac{1}{8}$	1.10	1.10	0.848	1.24	$4\frac{7}{16}$	—	2.95	2.13	1.13
HSS4×3×$\frac{3}{8}$	5.01	3.34	1.11	4.18	$2\frac{5}{16}$	—	10.6	6.59	1.07
×$\frac{5}{16}$	4.52	3.02	1.13	3.69	$2\frac{5}{8}$	—	9.41	5.75	1.08
×$\frac{1}{4}$	3.91	2.61	1.16	3.12	$2\frac{7}{8}$	—	7.96	4.81	1.10
×$\frac{3}{16}$	3.16	2.10	1.19	2.46	$3\frac{3}{16}$	—	6.26	3.74	1.12
×$\frac{1}{8}$	2.27	1.51	1.21	1.73	$3\frac{7}{16}$	—	4.38	2.59	1.13
HSS4×2$\frac{1}{2}$×$\frac{3}{8}$	3.17	2.54	0.922	3.20	$2\frac{5}{16}$	—	7.57	5.32	0.983
×$\frac{5}{16}$	2.89	2.32	0.947	2.85	$2\frac{5}{8}$	—	6.77	4.67	1.00
×$\frac{1}{4}$	2.53	2.02	0.973	2.43	$2\frac{7}{8}$	—	5.78	3.93	1.02
×$\frac{3}{16}$	2.06	1.65	0.999	1.93	$3\frac{1}{8}$	—	4.59	3.08	1.03
×$\frac{1}{8}$	1.49	1.19	1.03	1.36	$3\frac{7}{16}$	—	3.23	2.14	1.05
HSS4×2×$\frac{3}{8}$	1.80	1.80	0.729	2.31	$2\frac{5}{16}$	—	4.83	4.04	0.900
×$\frac{5}{16}$	1.67	1.67	0.754	2.08	$2\frac{5}{8}$	—	4.40	3.59	0.917
×$\frac{1}{4}$	1.48	1.48	0.779	1.79	$2\frac{7}{8}$	—	3.82	3.05	0.933
×$\frac{3}{16}$	1.22	1.22	0.804	1.43	$3\frac{3}{16}$	—	3.08	2.41	0.950
×$\frac{1}{8}$	0.898	0.898	0.830	1.02	$3\frac{7}{16}$	—	2.20	1.69	0.967
HSS3$\frac{1}{2}$×2$\frac{1}{2}$×$\frac{3}{8}$	2.77	2.21	0.904	2.82	—	—	6.16	4.57	0.900
×$\frac{5}{16}$	2.54	2.03	0.930	2.52	$2\frac{1}{8}$	—	5.53	4.03	0.917
×$\frac{1}{4}$	2.23	1.78	0.956	2.16	$2\frac{3}{8}$	—	4.75	3.40	0.933
×$\frac{3}{16}$	1.82	1.46	0.983	1.72	$2\frac{11}{16}$	—	3.78	2.67	0.950
×$\frac{1}{8}$	1.33	1.06	1.01	1.22	$2\frac{15}{16}$	—	2.67	1.87	0.967
HSS3$\frac{1}{2}$×2×$\frac{1}{4}$	1.30	1.30	0.766	1.58	$2\frac{3}{8}$	—	3.16	2.64	0.850
×$\frac{3}{16}$	1.08	1.08	0.792	1.27	$2\frac{11}{16}$	—	2.55	2.09	0.867
×$\frac{1}{8}$	0.795	0.795	0.818	0.912	$2\frac{15}{16}$	—	1.83	1.47	0.883

—Flat depth or width is too small to establish a workable flat.

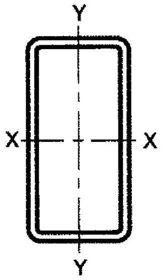


Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

Shape	Design Wall Thick-ness, *t*	Nominal Wt.	Area, *A*	*b/t*	*h/t*	Axis X-X			
						I	*S*	*r*	*Z*
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}
HSS3½×1½×¼	0.233	7.08	1.97	3.44	12.0	2.55	1.46	1.14	1.98
×3/16	0.174	5.59	1.54	5.62	17.1	2.12	1.21	1.17	1.60
×⅛	0.116	3.90	1.07	9.93	27.2	1.57	0.896	1.21	1.15
HSS3×2½×5/16	0.291	9.47	2.64	5.59	7.31	2.92	1.94	1.05	2.51
×¼	0.233	7.93	2.21	7.73	9.88	2.57	1.72	1.08	2.16
×3/16	0.174	6.23	1.71	11.4	14.2	2.11	1.41	1.11	1.73
×⅛	0.116	4.32	1.19	18.6	22.9	1.54	1.03	1.14	1.23
HSS3×2×5/16	0.291	8.41	2.35	3.87	7.31	2.38	1.59	1.01	2.11
×¼	0.233	7.08	1.97	5.58	9.88	2.13	1.42	1.04	1.83
×3/16	0.174	5.59	1.54	8.49	14.2	1.77	1.18	1.07	1.48
×⅛	0.116	3.90	1.07	14.2	22.9	1.30	0.867	1.10	1.06
HSS3×1½×¼	0.233	6.23	1.74	3.44	9.88	1.68	1.12	0.982	1.51
×3/16	0.174	4.95	1.37	5.62	14.2	1.42	0.945	1.02	1.24
×⅛	0.116	3.47	0.956	9.93	22.9	1.06	0.706	1.05	0.895
HSS3×1×3/16	0.174	4.31	1.19	2.75	14.2	1.07	0.713	0.947	0.989
×⅛	0.116	3.04	0.840	5.62	22.9	0.817	0.545	0.987	0.728
HSS2½×2×¼	0.233	6.23	1.74	5.58	7.73	1.33	1.06	0.874	1.37
×3/16	0.174	4.95	1.37	8.49	11.4	1.12	0.894	0.904	1.12
×⅛	0.116	3.47	0.956	14.2	18.6	0.833	0.667	0.934	0.809
HSS2½×1½×¼	0.233	5.38	1.51	3.44	7.73	1.03	0.822	0.826	1.11
×3/16	0.174	4.31	1.19	5.62	11.4	0.882	0.705	0.860	0.915
×⅛	0.116	3.04	0.840	9.93	18.6	0.668	0.535	0.892	0.671
HSS2½×1×3/16	0.174	3.67	1.02	2.75	11.4	0.646	0.517	0.796	0.713
×⅛	0.116	2.62	0.724	5.62	18.6	0.503	0.403	0.834	0.532
HSS2¼×2×3/16	0.174	4.63	1.28	8.49	9.93	0.859	0.764	0.819	0.952
×⅛	0.116	3.26	0.898	14.2	16.4	0.646	0.574	0.848	0.693
HSS2×1½×3/16	0.174	3.67	1.02	5.62	8.49	0.495	0.495	0.697	0.639
×⅛	0.116	2.62	0.724	9.93	14.2	0.383	0.383	0.728	0.475
HSS2×1×3/16	0.174	3.03	0.845	2.75	8.49	0.350	0.350	0.643	0.480
×⅛	0.116	2.19	0.608	5.62	14.2	0.280	0.280	0.679	0.366

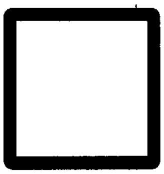
Note: For compactness criteria, refer to the end of Table 1-12.

Table 1-11 (continued)
Rectangular HSS
Dimensions and Properties

HSS$3\frac{1}{2}$–HSS2

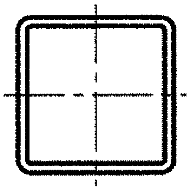
Shape	Axis Y-Y				Workable Flat		Torsion		Surface Area
	I	*S*	*r*	*Z*	Depth	Width	*J*	*C*	
	in.^{4}	in.^{3}	in.	in.^{3}	in.	in.	in.^{4}	in.^{3}	ft^2/ft
HSS$3\frac{1}{2}\times1\frac{1}{2}\times\frac{1}{4}$	0.638	0.851	0.569	1.06	$2\frac{3}{8}$	—	1.79	1.88	0.767
$\times\frac{3}{16}$	0.544	0.725	0.594	0.867	$2\frac{11}{16}$	—	1.49	1.51	0.784
$\times\frac{1}{8}$	0.411	0.548	0.619	0.630	$2\frac{15}{16}$	—	1.09	1.08	0.800
HSS$3\times2\frac{1}{2}\times\frac{5}{16}$	2.18	1.74	0.908	2.20	—	—	4.34	3.39	0.833
$\times\frac{1}{4}$	1.93	1.54	0.935	1.90	—	—	3.74	2.87	0.850
$\times\frac{3}{16}$	1.59	1.27	0.963	1.52	$2\frac{3}{16}$	—	3.00	2.27	0.867
$\times\frac{1}{8}$	1.16	0.931	0.990	1.09	$2\frac{7}{16}$	—	2.13	1.59	0.883
HSS$3\times2\times\frac{5}{16}$	1.24	1.24	0.725	1.58	—	—	2.87	2.60	0.750
$\times\frac{1}{4}$	1.11	1.11	0.751	1.38	—	—	2.52	2.23	0.767
$\times\frac{3}{16}$	0.932	0.932	0.778	1.12	$2\frac{3}{16}$	—	2.05	1.78	0.784
$\times\frac{1}{8}$	0.692	0.692	0.804	0.803	$2\frac{7}{16}$	—	1.47	1.25	0.800
HSS$3\times1\frac{1}{2}\times\frac{1}{4}$	0.543	0.725	0.559	0.911	$1\frac{7}{8}$	—	1.44	1.58	0.683
$\times\frac{3}{16}$	0.467	0.622	0.584	0.752	$2\frac{3}{16}$	—	1.21	1.28	0.700
$\times\frac{1}{8}$	0.355	0.474	0.610	0.550	$2\frac{7}{16}$	—	0.886	0.920	0.717
HSS$3\times1\times\frac{3}{16}$	0.173	0.345	0.380	0.432	$2\frac{3}{16}$	—	0.526	0.792	0.617
$\times\frac{1}{8}$	0.138	0.276	0.405	0.325	$2\frac{7}{16}$	—	0.408	0.585	0.633
HSS$2\frac{1}{2}\times2\times\frac{1}{4}$	0.930	0.930	0.731	1.17	—	—	1.90	1.82	0.683
$\times\frac{3}{16}$	0.786	0.786	0.758	0.956	—	—	1.55	1.46	0.700
$\times\frac{1}{8}$	0.589	0.589	0.785	0.694	—	—	1.12	1.04	0.717
HSS$2\frac{1}{2}\times1\frac{1}{2}\times\frac{1}{4}$	0.449	0.599	0.546	0.764	—	—	1.10	1.29	0.600
$\times\frac{3}{16}$	0.390	0.520	0.572	0.636	—	—	0.929	1.05	0.617
$\times\frac{1}{8}$	0.300	0.399	0.597	0.469	—	—	0.687	0.759	0.633
HSS$2\frac{1}{2}\times1\times\frac{3}{16}$	0.143	0.285	0.374	0.360	—	—	0.412	0.648	0.534
$\times\frac{1}{8}$	0.115	0.230	0.399	0.274	—	—	0.322	0.483	0.550
HSS$2\frac{1}{4}\times2\times\frac{3}{16}$	0.713	0.713	0.747	0.877	—	—	1.32	1.30	0.659
$\times\frac{1}{8}$	0.538	0.538	0.774	0.639	—	—	0.957	0.927	0.675
HSS$2\times1\frac{1}{2}\times\frac{3}{16}$	0.313	0.417	0.554	0.521	—	—	0.664	0.822	0.534
$\times\frac{1}{8}$	0.244	0.325	0.581	0.389	—	—	0.496	0.599	0.550
HSS$2\times1\times\frac{3}{16}$	0.112	0.225	0.365	0.288	—	—	0.301	0.505	0.450
$\times\frac{1}{8}$	0.0922	0.184	0.390	0.223	—	—	0.238	0.380	0.467

—Flat depth or width is too small to establish a workable flat.



HSS16–HSS8

Table 1–12
Square HSS
Dimensions and Properties



Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b*/*t*	*h*/*t*	*I*	*S*	*r*	*Z*	Workable Flat	Torsion		Surface Area
											J	*C*	
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS16×16×$\frac{5}{8}$	0.581	127.00	35.0	24.5	24.5	1370	171	6.25	200	$13\frac{3}{16}$	2170	276	5.17
×$\frac{1}{2}$	0.465	103.00	28.3	31.4	31.4	1130	141	6.31	164	$13\frac{3}{4}$	1770	224	5.20
×$\frac{3}{8}$	0.349	78.45	21.5	42.8	42.8	873	109	6.37	126	$14\frac{5}{16}$	1350	171	5.23
×$\frac{5}{16}$	0.291	65.82	18.1	52.0	52.0	739	92.3	6.39	106	$14\frac{5}{8}$	1140	144	5.25
HSS14×14×$\frac{5}{8}$	0.581	110.00	30.3	21.1	21.1	897	128	5.44	151	$11\frac{3}{16}$	1430	208	4.50
×$\frac{1}{2}$	0.465	89.55	24.6	27.1	27.1	743	106	5.49	124	$11\frac{3}{4}$	1170	170	4.53
×$\frac{3}{8}$	0.349	68.24	18.7	37.1	37.1	577	82.5	5.55	95.4	$12\frac{5}{16}$	900	130	4.57
×$\frac{5}{16}$	0.291	57.31	15.7	45.1	45.1	490	69.9	5.58	80.5	$12\frac{5}{8}$	759	109	4.58
HSS12×12×$\frac{5}{8}$	0.581	93.14	25.7	17.7	17.7	548	91.4	4.62	109	$9\frac{3}{16}$	885	151	3.83
×$\frac{1}{2}$	0.465	75.94	20.9	22.8	22.8	457	76.2	4.68	89.6	$9\frac{3}{4}$	728	123	3.87
×$\frac{3}{8}$	0.349	58.03	16.0	31.4	31.4	357	59.5	4.73	69.2	$10\frac{5}{16}$	561	94.6	3.90
×$\frac{5}{16}$	0.291	48.81	13.4	38.2	38.2	304	50.7	4.76	58.6	$10\frac{5}{8}$	474	79.7	3.92
×$\frac{1}{4}$	0.233	39.40	10.8	48.5	48.5	248	41.4	4.79	47.6	$10\frac{7}{8}$	384	64.5	3.93
×$\frac{3}{16}$	0.174	29.82	8.15	66.0	66.0	189	31.5	4.82	36.0	$11\frac{3}{16}$	290	48.6	3.95
HSS10×10×$\frac{5}{8}$	0.581	76.13	21.0	14.2	14.2	304	60.8	3.80	73.2	$7\frac{3}{16}$	498	102	3.17
×$\frac{1}{2}$	0.465	62.33	17.2	18.5	18.5	256	51.2	3.86	60.7	$7\frac{3}{4}$	412	84.2	3.20
×$\frac{3}{8}$	0.349	47.82	13.2	25.7	25.7	202	40.4	3.92	47.2	$8\frac{5}{16}$	320	64.8	3.23
×$\frac{5}{16}$	0.291	40.30	11.1	31.4	31.4	172	34.5	3.94	40.1	$8\frac{5}{8}$	271	54.8	3.25
×$\frac{1}{4}$	0.233	32.60	8.96	39.9	39.9	141	28.3	3.97	32.7	$8\frac{7}{8}$	220	44.4	3.27
×$\frac{3}{16}$	0.174	24.72	6.76	54.5	54.5	108	21.6	4.00	24.8	$9\frac{3}{16}$	167	33.6	3.28
HSS9×9×$\frac{5}{8}$	0.581	67.62	18.7	12.5	12.5	216	47.9	3.40	58.1	$6\frac{3}{16}$	356	81.6	2.83
×$\frac{1}{2}$	0.465	55.53	15.3	16.4	16.4	183	40.6	3.45	48.4	$6\frac{3}{4}$	296	67.4	2.87
×$\frac{3}{8}$	0.349	42.72	11.8	22.8	22.8	145	32.2	3.51	37.8	$7\frac{5}{16}$	231	52.1	2.90
×$\frac{5}{16}$	0.291	36.05	9.92	27.9	27.9	124	27.6	3.54	32.1	$7\frac{5}{8}$	196	44.0	2.92
×$\frac{1}{4}$	0.233	29.19	8.03	35.6	35.6	102	22.7	3.56	26.2	$7\frac{7}{8}$	159	35.8	2.93
×$\frac{3}{16}$	0.174	22.16	6.06	48.7	48.7	78.2	17.4	3.59	20.0	$8\frac{3}{16}$	121	27.1	2.95
×$\frac{1}{8}$	0.116	14.95	4.09	74.6	74.6	53.5	11.9	3.62	13.6	$8\frac{7}{16}$	82.0	18.3	2.97
HSS8×8×$\frac{5}{8}$	0.581	59.11	16.4	10.8	10.8	146	36.5	2.99	44.7	$5\frac{3}{16}$	244	63.2	2.50
×$\frac{1}{2}$	0.465	48.72	13.5	14.2	14.2	125	31.2	3.04	37.5	$5\frac{3}{4}$	204	52.4	2.53
×$\frac{3}{8}$	0.349	37.61	10.4	19.9	19.9	100	24.9	3.10	29.4	$6\frac{5}{16}$	160	40.7	2.57
×$\frac{5}{16}$	0.291	31.79	8.76	24.5	24.5	85.6	21.4	3.13	25.1	$6\frac{5}{8}$	136	34.5	2.58
×$\frac{1}{4}$	0.233	25.79	7.10	31.3	31.3	70.7	17.7	3.15	20.5	$6\frac{7}{8}$	111	28.1	2.60
×$\frac{3}{16}$	0.174	19.61	5.37	43.0	43.0	54.4	13.6	3.18	15.7	$7\frac{3}{16}$	84.5	21.3	2.62
×$\frac{1}{8}$	0.116	13.25	3.62	66.0	66.0	37.4	9.34	3.21	10.7	$7\frac{7}{16}$	57.3	14.4	2.63

Note: For compactness criteria, refer to the end of Table 1-12.

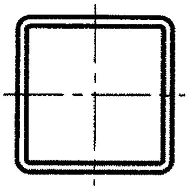
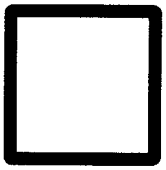


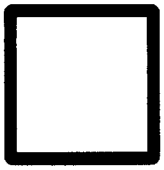
Table 1–12 (continued)
Square HSS
Dimensions and Properties



HSS7–HSS4½

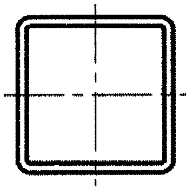
Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b*/*t*	*h*/*t*	*I*	*S*	*r*	*Z*	Workable Flat	Torsion		Surface Area
											J	*C*	
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS7×7×5/8	0.581	50.60	14.0	9.05	9.05	93.4	26.7	2.58	33.1	4 3/16	158	47.1	2.17
×1/2	0.465	41.91	11.6	12.1	12.1	80.5	23.0	2.63	27.9	4 3/4	133	39.3	2.20
×3/8	0.349	32.51	8.97	17.1	17.1	65.0	18.6	2.69	22.1	5 5/16	105	30.7	2.23
×5/16	0.291	27.54	7.59	21.1	21.1	56.1	16.0	2.72	18.9	5 5/8	89.7	26.1	2.25
×1/4	0.233	22.39	6.17	27.0	27.0	46.5	13.3	2.75	15.5	5 7/8	73.5	21.3	2.27
×3/16	0.174	17.06	4.67	37.2	37.2	36.0	10.3	2.77	11.9	6 3/16	56.1	16.2	2.28
×1/8	0.116	11.55	3.16	57.3	57.3	24.8	7.09	2.80	8.13	6 7/16	38.2	11.0	2.30
HSS6×6×5/8	0.581	42.10	11.7	7.33	7.33	55.2	18.4	2.17	23.2	3 3/16	94.9	33.4	1.83
×1/2	0.465	35.11	9.74	9.90	9.90	48.3	16.1	2.23	19.8	3 3/4	81.1	28.1	1.87
×3/8	0.349	27.41	7.58	14.2	14.2	39.5	13.2	2.28	15.8	4 5/16	64.6	22.1	1.90
×5/16	0.291	23.29	6.43	17.6	17.6	34.3	11.4	2.31	13.6	4 5/8	55.4	18.9	1.92
×1/4	0.233	18.99	5.24	22.8	22.8	28.6	9.54	2.34	11.2	4 7/8	45.6	15.4	1.93
×3/16	0.174	14.51	3.98	31.5	31.5	22.3	7.42	2.37	8.63	5 3/16	35.0	11.8	1.95
×1/8	0.116	9.85	2.70	48.7	48.7	15.5	5.15	2.39	5.92	5 7/16	23.9	8.03	1.97
HSS5½×5½×3/8	0.349	24.85	6.88	12.8	12.8	29.7	10.8	2.08	13.1	3 13/16	49.0	18.4	1.73
×5/16	0.291	21.16	5.85	15.9	15.9	25.9	9.43	2.11	11.3	4 1/8	42.2	15.7	1.75
×1/4	0.233	17.28	4.77	20.6	20.6	21.7	7.90	2.13	9.32	4 3/8	34.8	12.9	1.77
×3/16	0.174	13.23	3.63	28.6	28.6	17.0	6.17	2.16	7.19	4 11/16	26.7	9.85	1.78
×1/8	0.116	9.00	2.46	44.4	44.4	11.8	4.30	2.19	4.95	4 15/16	18.3	6.72	1.80
HSS5×5×1/2	0.465	28.30	7.88	7.75	7.75	26.0	10.4	1.82	13.1	2 3/4	44.6	18.7	1.53
×3/8	0.349	22.30	6.18	11.3	11.3	21.7	8.68	1.87	10.6	3 5/16	36.1	14.9	1.57
×5/16	0.291	19.03	5.26	14.2	14.2	19.0	7.62	1.90	9.16	3 5/8	31.2	12.8	1.58
×1/4	0.233	15.58	4.30	18.5	18.5	16.0	6.41	1.93	7.61	3 7/8	25.8	10.5	1.60
×3/16	0.174	11.96	3.28	25.7	25.7	12.6	5.03	1.96	5.89	4 3/16	19.9	8.08	1.62
×1/8	0.116	8.15	2.23	40.1	40.1	8.80	3.52	1.99	4.07	4 7/16	13.7	5.53	1.63
HSS4½×4½×1/2	0.465	24.90	6.95	6.68	6.68	18.1	8.03	1.61	10.2	2 1/4	31.3	14.8	1.37
×3/8	0.349	19.75	5.48	9.89	9.89	15.3	6.79	1.67	8.36	2 13/16	25.7	11.9	1.40
×5/16	0.291	16.91	4.68	12.5	12.5	13.5	6.00	1.70	7.27	3 1/8	22.3	10.2	1.42
×1/4	0.233	13.88	3.84	16.3	16.3	11.4	5.08	1.73	6.06	3 3/8	18.5	8.44	1.43
×3/16	0.174	10.68	2.93	22.9	22.9	9.02	4.01	1.75	4.71	3 11/16	14.4	6.49	1.45
×1/8	0.116	7.30	2.00	35.8	35.8	6.35	2.82	1.78	3.27	3 15/16	9.92	4.45	1.47

Note: For compactness criteria, refer to the end of Table 1-12.



HSS4–HSS2

Table 1–12 (continued)
Square HSS
Dimensions and Properties



Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*b*/*t*	*h*/*t*	*I*	*S*	*r*	*Z*	Workable Flat	Torsion		Surface Area
											J	*C*	
	in.	lb/ft	in.^{2}			in.^{4}	in.^{3}	in.	in.^{3}	in.	in.^{4}	in.^{3}	ft^2/ft
HSS4×4×1/2	0.465	21.50	6.02	5.60	5.60	11.9	5.97	1.41	7.70	—	21.0	11.2	1.20
×3/8	0.349	17.20	4.78	8.46	8.46	10.3	5.13	1.47	6.39	2 5/16	17.5	9.14	1.23
×5/16	0.291	14.78	4.10	10.7	10.7	9.14	4.57	1.49	5.59	2 5/8	15.3	7.91	1.25
×1/4	0.233	12.18	3.37	14.2	14.2	7.80	3.90	1.52	4.69	2 7/8	12.8	6.56	1.27
×3/16	0.174	9.40	2.58	20.0	20.0	6.21	3.10	1.55	3.67	3 3/16	10.0	5.07	1.28
×1/8	0.116	6.45	1.77	31.5	31.5	4.40	2.20	1.58	2.56	3 7/16	6.91	3.49	1.30
HSS3 1/2×3 1/2×3/8	0.349	14.65	4.09	7.03	7.03	6.49	3.71	1.26	4.69	—	11.2	6.77	1.07
×5/16	0.291	12.65	3.52	9.03	9.03	5.84	3.34	1.29	4.14	2 1/8	9.89	5.90	1.08
×1/4	0.233	10.48	2.91	12.0	12.0	5.04	2.88	1.32	3.50	2 3/8	8.35	4.92	1.10
×3/16	0.174	8.13	2.24	17.1	17.1	4.05	2.31	1.35	2.76	2 11/16	6.56	3.83	1.12
×1/8	0.116	5.60	1.54	27.2	27.2	2.90	1.66	1.37	1.93	2 15/16	4.58	2.65	1.13
HSS3×3×3/8	0.349	12.09	3.39	5.60	5.60	3.78	2.52	1.06	3.25	—	6.64	4.74	0.900
×5/16	0.291	10.53	2.94	7.31	7.31	3.45	2.30	1.08	2.90	—	5.94	4.18	0.917
×1/4	0.233	8.78	2.44	9.88	9.88	3.02	2.01	1.11	2.48	—	5.08	3.52	0.933
×3/16	0.174	6.85	1.89	14.2	14.2	2.46	1.64	1.14	1.97	2 3/16	4.03	2.76	0.950
×1/8	0.116	4.75	1.30	22.9	22.9	1.78	1.19	1.17	1.40	2 7/16	2.84	1.92	0.967
HSS2 1/2×2 1/2×5/16	0.291	8.40	2.35	5.59	5.59	1.82	1.46	0.880	1.88	—	3.20	2.74	0.750
×1/4	0.233	7.08	1.97	7.73	7.73	1.63	1.30	0.908	1.63	—	2.79	2.35	0.767
×3/16	0.174	5.57	1.54	11.4	11.4	1.35	1.08	0.937	1.32	—	2.25	1.86	0.784
×1/8	0.116	3.90	1.07	18.6	18.6	0.998	0.799	0.965	0.947	—	1.61	1.31	0.800
HSS2 1/4×2 1/4×1/4	0.233	6.23	1.74	6.66	6.66	1.13	1.01	0.806	1.28	—	1.96	1.85	0.683
×3/16	0.174	4.94	1.37	9.93	9.93	0.953	0.847	0.835	1.04	—	1.60	1.48	0.700
×1/8	0.116	3.47	0.956	16.4	16.4	0.712	0.633	0.863	0.755	—	1.15	1.05	0.717
HSS2×2×1/4	0.233	5.38	1.51	5.58	5.58	0.747	0.747	0.704	0.964	—	1.31	1.41	0.600
×3/16	0.174	4.30	1.19	8.49	8.49	0.641	0.641	0.733	0.797	—	1.09	1.14	0.617
×1/8	0.116	3.04	0.840	14.2	14.2	0.486	0.486	0.761	0.584	—	0.796	0.817	0.633

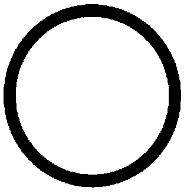
— Flat depth or width is too small to establish a workable flat.

Rectangular and Square HSS

Compactness Criteria

Nominal Wall Thickness	Compactness Criteria for Rectangular and Square HSS			
	Compression	Flexure		Shear
	non-slender up to	compact up to	compact up to	$C_v = 1.0$ up to
	Flange Width	Flange Width	Web Width	Web Depth
5/8	20	18	20	20
1/2	16	14	20	20
3/8	12	10	20	20
5/16	10	9	18	18
1/4	8	7	14	14
3/16	6	5	10	10
1/8	4	3½	7	7

Note: Compactness criteria given for $F_y = 46$ ksi.



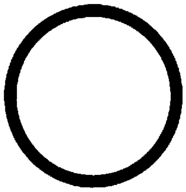
HSS20.000–
HSS10.000

Table 1–13
Round HSS
Dimensions and Properties

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	D/t	I	S	r	Z	Torsion J	Torsion C
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS20.000×0.500	0.465	104.00	28.5	43.0	1360	136	6.91	177	2720	272
×0.375[f]	0.349	78.67	21.5	57.3	1040	104	6.95	135	2080	208
HSS18.000×0.500	0.465	93.54	25.6	38.7	985	109	6.20	143	1970	219
×0.375[f]	0.349	70.66	19.4	51.6	754	83.8	6.24	109	1510	168
HSS16.000×0.625	0.581	103.00	28.1	27.5	838	105	5.46	138	1680	209
×0.500	0.465	82.85	22.7	34.4	685	85.7	5.49	112	1370	171
×0.438	0.407	72.87	19.9	39.3	606	75.8	5.51	99.0	1210	152
×0.375	0.349	62.64	17.2	45.8	526	65.7	5.53	85.5	1050	131
×0.312[f]	0.291	52.32	14.4	55.0	443	55.4	5.55	71.8	886	111
×0.250[f]	0.233	42.09	11.5	68.7	359	44.8	5.58	57.9	717	89.7
HSS14.000×0.625	0.581	89.36	24.5	24.1	552	78.9	4.75	105	1100	158
×0.500	0.465	72.16	19.8	30.1	453	64.8	4.79	85.2	907	130
×0.375	0.349	54.62	15.0	40.1	349	49.8	4.83	65.1	698	100
×0.312	0.291	45.65	12.5	48.1	295	42.1	4.85	54.7	589	84.2
×0.250[f]	0.233	36.75	10.1	60.1	239	34.1	4.87	44.2	478	68.2
HSS12.750×0.500	0.465	65.48	17.9	27.4	339	53.2	4.35	70.2	678	106
×0.375	0.349	49.61	13.6	36.5	262	41.0	4.39	53.7	523	82.1
×0.250[f]	0.233	33.41	9.16	54.7	180	28.2	4.43	36.5	359	56.3
HSS10.750×0.500	0.465	54.79	15.0	23.1	199	37.0	3.64	49.2	398	74.1
×0.375	0.349	41.59	11.4	30.8	154	28.7	3.68	37.8	309	57.4
×0.250	0.233	28.06	7.70	46.1	106	19.8	3.72	25.8	213	39.6
HSS10.000×0.625	0.581	62.64	17.2	17.2	191	38.3	3.34	51.6	383	76.6
×0.500	0.465	50.78	13.9	21.5	159	31.7	3.38	42.3	317	63.5
×0.375	0.349	38.58	10.6	28.7	123	24.7	3.41	32.5	247	49.3
×0.312	0.291	32.31	8.88	34.4	105	20.9	3.43	27.4	209	41.9
×0.250	0.233	26.06	7.15	42.9	85.3	17.1	3.45	22.2	171	34.1
×0.188[f]	0.174	19.72	5.37	57.5	64.8	13.0	3.47	16.8	130	25.9

[f] Shape exceeds compact limit for flexure with $F_y = 42$ ksi.

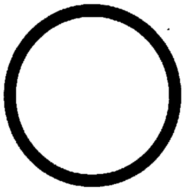
Table 1-13 (continued)
Round HSS
Dimensions and Properties



HSS9.625–HSS6.875

Shape	Design Wall Thickness, t	Nominal Wt.	Area, A	D/t	I	S	r	Z	Torsion J	Torsion C
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS9.625×0.500	0.465	48.77	13.4	20.7	141	29.2	3.24	39.0	281	58.5
×0.375	0.349	37.08	10.2	27.6	110	22.8	3.28	30.0	219	45.5
×0.312	0.291	31.06	8.53	33.1	93.0	19.3	3.30	25.4	186	38.7
×0.250	0.233	25.06	6.87	41.3	75.9	15.8	3.32	20.6	152	31.5
×0.188[f]	0.174	18.97	5.17	55.3	57.7	12.0	3.34	15.5	115	24.0
HSS8.625×0.625	0.581	53.45	14.7	14.8	119	27.7	2.85	37.7	239	55.4
×0.500	0.465	43.43	11.9	18.5	100	23.1	2.89	31.0	199	46.2
×0.375	0.349	33.07	9.07	24.7	77.8	18.0	2.93	23.9	156	36.1
×0.322	0.300	28.58	7.85	28.8	68.1	15.8	2.95	20.8	136	31.6
×0.250	0.233	22.38	6.14	37.0	54.1	12.5	2.97	16.4	108	25.1
×0.188[f]	0.174	16.96	4.62	49.6	41.3	9.57	2.99	12.4	82.5	19.1
HSS7.625×0.375	0.349	29.06	7.98	21.8	52.9	13.9	2.58	18.5	106	27.8
×0.328	0.305	25.59	7.01	25.0	47.1	12.3	2.59	16.4	94.1	24.7
HSS7.50×0.500	0.465	37.42	10.3	16.1	63.9	17.0	2.49	23.0	128	34.1
×0.375	0.349	28.56	7.84	21.5	50.2	13.4	2.53	17.9	100	26.8
×0.312	0.291	23.97	6.59	25.8	42.9	11.4	2.55	15.1	85.8	22.9
×0.250	0.233	19.38	5.32	32.2	35.2	9.37	2.57	12.3	70.3	18.7
×0.188	0.174	14.70	4.00	43.1	26.9	7.17	2.59	9.34	53.8	14.3
HSS7.000×0.500	0.465	34.74	9.55	15.1	51.2	14.6	2.32	19.9	102	29.3
×0.375	0.349	26.56	7.29	20.1	40.4	11.6	2.35	15.5	80.9	23.1
×0.312	0.291	22.31	6.13	24.1	34.6	9.88	2.37	13.1	69.1	19.8
×0.250	0.233	18.04	4.95	30.0	28.4	8.11	2.39	10.7	56.8	16.2
×0.188	0.174	13.69	3.73	40.2	21.7	6.21	2.41	8.11	43.5	12.4
×0.125[f]	0.116	9.19	2.51	60.3	14.9	4.25	2.43	5.50	29.7	8.49
HSS6.875×0.500	0.465	34.07	9.36	14.8	48.3	14.1	2.27	19.1	96.7	28.1
×0.375	0.349	26.06	7.16	19.7	38.2	11.1	2.31	14.9	76.4	22.2
×0.312	0.291	21.89	6.02	23.6	32.7	9.51	2.33	12.6	65.4	19.0
×0.250	0.233	17.71	4.86	29.5	26.8	7.81	2.35	10.3	53.7	15.6
×0.188	0.174	13.44	3.66	39.5	20.6	5.99	2.37	7.81	41.1	12.0

[f] Shape exceeds compact limit for flexure with $F_y = 42$ ksi.



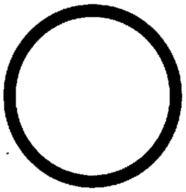
HSS6.625–
HSS5.000

Table 1–13 (continued)
Round HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*D/t*	*I*	*S*	*r*	*Z*	Torsion	
									J	*C*
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS6.625×0.500	0.465	32.74	9.00	14.2	42.9	13.0	2.18	17.7	85.9	25.9
×0.432	0.402	28.60	7.86	16.5	38.2	11.5	2.20	15.6	76.4	23.1
×0.375	0.349	25.06	6.88	19.0	34.0	10.3	2.22	13.8	68.0	20.5
×0.312	0.291	21.06	5.79	22.8	29.1	8.79	2.24	11.7	58.2	17.6
×0.280	0.260	18.99	5.20	25.5	26.4	7.96	2.25	10.5	52.7	15.9
×0.250	0.233	17.04	4.68	28.4	23.9	7.22	2.26	9.52	47.9	14.4
×0.188	0.174	12.94	3.53	38.1	18.4	5.54	2.28	7.24	36.7	11.1
×0.125[f]	0.116	8.69	2.37	57.1	12.6	3.79	2.30	4.92	25.1	7.59
HSS6.000×0.500	0.465	29.40	8.09	12.9	31.2	10.4	1.96	14.3	62.4	20.8
×0.375	0.349	22.55	6.20	17.2	24.8	8.28	2.00	11.2	49.7	16.6
×0.312	0.291	18.97	5.22	20.6	21.3	7.11	2.02	9.49	42.6	14.2
×0.280	0.260	17.12	4.69	23.1	19.3	6.45	2.03	8.57	38.7	12.9
×0.250	0.233	15.37	4.22	25.8	17.6	5.86	2.04	7.75	35.2	11.7
×0.188	0.174	11.68	3.18	34.5	13.5	4.51	2.06	5.91	27.0	9.02
×0.125[f]	0.116	7.85	2.14	51.7	9.28	3.09	2.08	4.02	18.6	6.19
HSS5.563×0.500	0.465	27.06	7.45	12.0	24.4	8.77	1.81	12.1	48.8	17.5
×0.375	0.349	20.80	5.72	15.9	19.5	7.02	1.85	9.50	39.0	14.0
×0.258	0.240	14.63	4.01	23.2	14.2	5.12	1.88	6.80	28.5	10.2
×0.188	0.174	10.80	2.95	32.0	10.7	3.85	1.91	5.05	21.4	7.70
×0.134	0.124	7.78	2.12	44.9	7.84	2.82	1.92	3.67	15.7	5.64
HSS5.500×0.500	0.465	26.73	7.36	11.8	23.5	8.55	1.79	11.8	47.0	17.1
×0.375	0.349	20.55	5.65	15.8	18.8	6.84	1.83	9.27	37.6	13.7
×0.258	0.240	14.46	3.97	22.9	13.7	5.00	1.86	6.64	27.5	10.0
HSS5.000×0.500	0.465	24.05	6.62	10.8	17.2	6.88	1.61	9.60	34.4	13.8
×0.375	0.349	18.54	5.10	14.3	13.9	5.55	1.65	7.56	27.7	11.1
×0.312	0.291	15.64	4.30	17.2	12.0	4.79	1.67	6.46	24.0	9.58
×0.258	0.240	13.08	3.59	20.8	10.2	4.08	1.69	5.44	20.4	8.15
×0.250	0.233	12.69	3.49	21.5	9.94	3.97	1.69	5.30	19.9	7.95
×0.188	0.174	9.67	2.64	28.7	7.69	3.08	1.71	4.05	15.4	6.15
×0.125	0.116	6.51	1.78	43.1	5.31	2.12	1.73	2.77	10.6	4.25

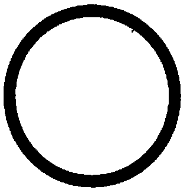
[f] Shape exceeds compact limit for flexure with F_y = 42 ksi.

Table 1–13 (continued)
Round HSS
Dimensions and Properties



HSS4.500–
HSS2.500

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*D/t*	*I*	*S*	*r*	*Z*	Torsion	
									J	*C*
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS4.500×0.375	0.349	16.54	4.55	12.9	9.87	4.39	1.47	6.03	19.7	8.78
×0.337	0.313	15.00	4.12	14.4	9.07	4.03	1.48	5.50	18.1	8.06
×0.237	0.220	10.80	2.96	20.5	6.79	3.02	1.52	4.03	13.6	6.04
×0.188	0.174	8.67	2.36	25.9	5.54	2.46	1.53	3.26	11.1	4.93
×0.125	0.116	5.85	1.60	38.8	3.84	1.71	1.55	2.23	7.68	3.41
HSS4.000×0.313	0.291	12.34	3.39	13.7	5.87	2.93	1.32	4.01	11.7	5.87
×0.250	0.233	10.00	2.76	17.2	4.91	2.45	1.33	3.31	9.82	4.91
×0.237	0.220	9.53	2.61	18.2	4.68	2.34	1.34	3.15	9.36	4.68
×0.226	0.210	9.12	2.50	19.0	4.50	2.25	1.34	3.02	9.01	4.50
×0.220	0.205	8.89	2.44	19.5	4.41	2.21	1.34	2.96	8.83	4.41
×0.188	0.174	7.66	2.09	23.0	3.83	1.92	1.35	2.55	7.67	3.83
×0.125	0.116	5.18	1.42	34.5	2.67	1.34	1.37	1.75	5.34	2.67
HSS3.500×0.313	0.291	10.66	2.93	12.0	3.81	2.18	1.14	3.00	7.61	4.35
×0.300	0.279	10.26	2.82	12.5	3.69	2.11	1.14	2.90	7.38	4.22
×0.250	0.233	8.69	2.39	15.0	3.21	1.83	1.16	2.49	6.41	3.66
×0.216	0.201	7.58	2.08	17.4	2.84	1.63	1.17	2.19	5.69	3.25
×0.203	0.189	7.15	1.97	18.5	2.70	1.54	1.17	2.07	5.41	3.09
×0.188	0.174	6.66	1.82	20.1	2.52	1.44	1.18	1.93	5.04	2.88
×0.125	0.116	4.51	1.23	30.2	1.77	1.01	1.20	1.33	3.53	2.02
HSS3.000×0.250	0.233	7.35	2.03	12.9	1.95	1.30	0.982	1.79	3.90	2.60
×0.216	0.201	6.43	1.77	14.9	1.74	1.16	0.992	1.58	3.48	2.32
×0.203	0.189	6.07	1.67	15.9	1.66	1.10	0.996	1.50	3.31	2.21
×0.188	0.174	5.65	1.54	17.2	1.55	1.03	1.00	1.39	3.10	2.06
×0.152	0.141	4.63	1.27	21.3	1.30	0.865	1.01	1.15	2.59	1.73
×0.134	0.124	4.11	1.12	24.2	1.16	0.774	1.02	1.03	2.32	1.55
×0.125	0.116	3.84	1.05	25.9	1.09	0.730	1.02	0.965	2.19	1.46
HSS2.875×0.250	0.233	7.02	1.93	12.3	1.70	1.18	0.938	1.63	3.40	2.37
×0.203	0.189	5.80	1.59	15.2	1.45	1.01	0.952	1.37	2.89	2.01
×0.188	0.174	5.40	1.48	16.5	1.35	0.941	0.957	1.27	2.70	1.88
×0.125	0.116	3.67	1.01	24.8	0.958	0.667	0.976	0.884	1.92	1.33
HSS2.500×0.250	0.233	6.01	1.66	10.7	1.08	0.862	0.806	1.20	2.15	1.72
×0.188	0.174	4.65	1.27	14.4	0.865	0.692	0.825	0.943	1.73	1.38
×0.125	0.116	3.17	0.869	21.6	0.619	0.495	0.844	0.660	1.24	0.990

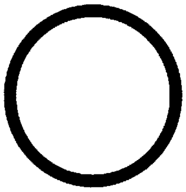


HSS2.375–
HSS1.660

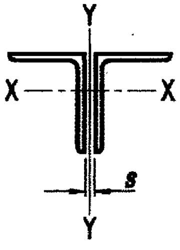
Table 1–13 (continued)
Round HSS
Dimensions and Properties

Shape	Design Wall Thickness, *t*	Nominal Wt.	Area, *A*	*D*/*t*	*I*	*S*	*r*	*Z*	Torsion	
									J	*C*
	in.	lb/ft	in.^{2}		in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}
HSS2.375×0.250	0.233	5.68	1.57	10.2	0.910	0.766	0.762	1.07	1.82	1.53
×0.218	0.203	5.03	1.39	11.7	0.824	0.694	0.771	0.960	1.65	1.39
×0.188	0.174	4.40	1.20	13.6	0.733	0.617	0.781	0.845	1.47	1.23
×0.154	0.143	3.66	1.00	16.6	0.627	0.528	0.791	0.713	1.25	1.06
×0.125	0.116	3.01	0.823	20.5	0.527	0.443	0.800	0.592	1.05	0.887
HSS1.900×0.188	0.174	3.44	0.943	10.9	0.355	0.374	0.613	0.520	0.710	0.747
×0.145	0.135	2.72	0.749	14.1	0.293	0.309	0.626	0.421	0.586	0.617
×0.120	0.111	2.28	0.624	17.1	0.251	0.264	0.634	0.356	0.501	0.527
HSS1.660×0.140	0.130	2.27	0.625	12.8	0.184	0.222	0.543	0.305	0.368	0.444

Table 1–14
Pipe
Dimensions and Properties

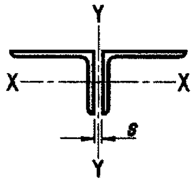


Shape	Nominal Wt.	Dimensions: Outside Diameter	Dimensions: Inside Diameter	Nominal Wall Thickness	Design Wall Thickness	Area	D/t	I	S	r	J	Z
	lb/ft	in.	in.	in.	in.	in.^{2}		in.^{4}	in.^{3}	in.	in.^{4}	in.^{3}
Standard Weight (Std.)												
Pipe 12 Std.	49.6	12.8	12.0	0.375	0.349	13.6	36.5	262	41.0	4.39	523	53.7
Pipe 10 Std.	40.5	10.8	10.0	0.365	0.340	11.1	31.6	151	28.1	3.68	302	36.9
Pipe 8 Std.	28.6	8.63	7.98	0.322	0.300	7.85	28.8	68.1	15.8	2.95	136	20.8
Pipe 6 Std.	19.0	6.63	6.07	0.280	0.261	5.22	25.4	26.5	7.99	2.25	52.9	10.6
Pipe 5 Std.	14.6	5.56	5.05	0.258	0.241	4.03	23.1	14.3	5.14	1.88	28.6	6.83
Pipe 4 Std.	10.8	4.50	4.03	0.237	0.221	2.97	20.4	6.82	3.03	1.51	13.6	4.05
Pipe 3½ Std.	9.12	4.00	3.55	0.226	0.211	2.51	19.0	4.52	2.26	1.34	9.04	3.03
Pipe 3 Std.	7.58	3.50	3.07	0.216	0.201	2.08	17.4	2.85	1.63	1.17	5.69	2.19
Pipe 2½ Std.	5.80	2.88	2.47	0.203	0.189	1.59	15.2	1.45	1.01	0.952	2.89	1.37
Pipe 2 Std.	3.66	2.38	2.07	0.154	0.143	1.00	16.6	0.627	0.528	0.791	1.25	0.713
Pipe 1½ Std.	2.72	1.90	1.61	0.145	0.135	0.750	14.1	0.293	0.309	0.626	0.586	0.421
Pipe 1¼ Std.	2.27	1.66	1.38	0.140	0.130	0.620	12.8	0.184	0.222	0.543	0.368	0.305
Pipe 1 Std.	1.68	1.32	1.05	0.133	0.124	0.460	10.6	0.0830	0.126	0.423	0.166	0.177
Pipe ¾ Std.	1.13	1.05	0.824	0.113	0.105	0.310	10.0	0.0350	0.0671	0.336	0.0700	0.0942
Pipe ½ Std.	0.850	0.840	0.622	0.109	0.101	0.230	8.32	0.0160	0.0388	0.264	0.0320	0.0555
Extra Strong (x-Strong)												
Pipe 12 x-Strong	65.5	12.8	11.8	0.500	0.465	17.9	27.4	339	53.2	4.35	678	70.2
Pipe 10 x-Strong	54.8	10.8	9.75	0.500	0.465	15.0	23.1	199	37.0	3.64	398	49.2
Pipe 8 x-Strong	43.4	8.63	7.63	0.500	0.465	11.9	18.5	100	23.1	2.89	199	31.0
Pipe 6 x-Strong	28.6	6.63	5.76	0.432	0.403	7.88	16.4	38.3	11.6	2.20	76.6	15.6
Pipe 5 x-Strong	20.8	5.56	4.81	0.375	0.349	5.72	15.9	19.5	7.02	1.85	39.0	9.50
Pipe 4 x-Strong	15.0	4.50	3.83	0.337	0.315	4.14	14.3	9.12	4.05	1.48	18.2	5.53
Pipe 3½ x-Strong	12.5	4.00	3.36	0.318	0.296	3.44	13.5	5.94	2.97	1.31	11.9	4.07
Pipe 3 x-Strong	10.3	3.50	2.90	0.300	0.280	2.83	12.5	3.70	2.11	1.14	7.40	2.91
Pipe 2½ x-Strong	7.67	2.88	2.32	0.276	0.257	2.11	11.2	1.83	1.27	0.930	3.66	1.77
Pipe 2 x-Strong	5.03	2.38	1.94	0.218	0.204	1.39	11.6	0.827	0.696	0.771	1.65	0.964
Pipe 1½ x-Strong	3.63	1.90	1.50	0.200	0.186	1.00	10.2	0.372	0.392	0.610	0.744	0.549
Pipe 1¼ x-Strong	3.00	1.66	1.28	0.191	0.178	0.830	9.33	0.231	0.278	0.528	0.462	0.393
Pipe 1 x-Strong	2.17	1.32	0.957	0.179	0.166	0.600	7.92	0.101	0.154	0.410	0.202	0.221
Pipe ¾ x-Strong	1.48	1.05	0.742	0.154	0.143	0.410	7.34	0.0430	0.0818	0.325	0.0860	0.119
Pipe ½ x-Strong	1.09	0.840	0.546	0.147	0.137	0.300	6.13	0.0190	0.0462	0.253	0.0380	0.0686
Double-Extra Strong (xx-Strong)												
Pipe 8 xx-Strong	72.5	8.63	6.88	0.875	0.816	20.0	10.6	154	35.8	2.78	308	49.9
Pipe 6 xx-Strong	53.2	6.63	4.90	0.864	0.805	14.7	8.23	63.5	19.2	2.08	127	27.4
Pipe 5 xx-Strong	38.6	5.56	4.06	0.750	0.699	10.7	7.96	32.2	11.6	1.74	64.4	16.7
Pipe 4 xx-Strong	27.6	4.50	3.15	0.674	0.628	7.64	7.17	14.7	6.53	1.39	29.4	9.50
Pipe 3 xx-Strong	18.6	3.50	2.30	0.600	0.559	5.16	6.26	5.79	3.31	1.06	11.6	4.89
Pipe 2½ xx-Strong	13.7	2.88	1.77	0.552	0.514	3.81	5.59	2.78	1.94	0.854	5.56	2.91
Pipe 2 xx-Strong	9.04	2.38	1.50	0.436	0.406	2.51	5.85	1.27	1.07	0.711	2.54	1.60



LLBB

Table 1–15
Double Angles
Properties



SLBB

Shape	Area	Axis Y-Y						LLBB			SLBB		
		Radius of Gyration						Q_s		r_x	Q_s		r_x
		LLBB			SLBB			Angles in Contact	Angles Separated		Angles in Contact	Angles Separated	
		Separation, s, in.			Separation, s, in.								
	in.^{2}	0	3/8	3/4	0	3/8	3/4			in.			in.
2L8×8×1 1/8	33.6	3.41	3.54	3.68	3.41	3.54	3.68	1.00	1.00	2.41	1.00	1.00	2.41
×1	30.2	3.39	3.52	3.66	3.39	3.52	3.66	1.00	1.00	2.43	1.00	1.00	2.43
×7/8	26.6	3.36	3.50	3.63	3.36	3.50	3.63	1.00	1.00	2.45	1.00	1.00	2.45
×3/4	23.0	3.34	3.47	3.61	3.34	3.47	3.61	1.00	1.00	2.46	1.00	1.00	2.46
×5/8	19.4	3.32	3.45	3.58	3.32	3.45	3.58	1.00	0.997	2.48	1.00	0.997	2.48
×9/16	17.5	3.31	3.44	3.57	3.31	3.44	3.57	1.00	0.959	2.49	1.00	0.959	2.49
×1/2	15.7	3.30	3.43	3.56	3.30	3.43	3.56	0.998	0.912	2.49	0.998	0.912	2.49
2L8×6×1	26.1	2.39	2.52	2.66	3.63	3.77	3.91	1.00	1.00	2.49	1.00	1.00	1.72
×7/8	23.1	2.37	2.50	2.63	3.61	3.75	3.89	1.00	1.00	2.50	1.00	1.00	1.74
×3/4	20.0	2.35	2.47	2.61	3.59	3.72	3.86	1.00	1.00	2.52	1.00	1.00	1.75
×5/8	16.8	2.33	2.45	2.59	3.57	3.70	3.84	1.00	0.997	2.54	1.00	0.997	1.77
×9/16	15.2	2.32	2.44	2.58	3.55	3.69	3.83	1.00	0.959	2.55	1.00	0.959	1.78
×1/2	13.6	2.31	2.43	2.56	3.54	3.68	3.81	1.00	0.912	2.55	0.998	0.912	1.79
×7/16	12.0	2.30	2.42	2.55	3.53	3.66	3.80	1.00	0.850	2.56	0.938	0.850	1.80
2L8×4×1	22.1	1.46	1.60	1.75	3.94	4.08	4.23	1.00	1.00	2.51	1.00	1.00	1.03
×7/8	19.6	1.44	1.57	1.72	3.91	4.06	4.21	1.00	1.00	2.53	1.00	1.00	1.04
×3/4	17.0	1.42	1.55	1.69	3.89	4.03	4.18	1.00	1.00	2.55	1.00	1.00	1.05
×5/8	14.3	1.39	1.52	1.66	3.86	4.00	4.15	1.00	0.997	2.56	1.00	0.997	1.06
×9/16	13.0	1.38	1.51	1.65	3.85	3.99	4.13	1.00	0.959	2.57	1.00	0.959	1.07
×1/2	11.6	1.38	1.50	1.63	3.83	3.97	4.12	1.00	0.912	2.58	0.998	0.912	1.08
×7/16	10.2	1.37	1.49	1.62	3.82	3.96	4.10	1.00	0.850	2.59	0.938	0.850	1.09
2L7×4×3/4	15.4	1.48	1.61	1.75	3.34	3.48	3.63	1.00	1.00	2.21	1.00	1.00	1.08
×5/8	13.0	1.45	1.58	1.73	3.31	3.46	3.60	1.00	1.00	2.23	1.00	1.00	1.10
×1/2	10.5	1.44	1.56	1.70	3.29	3.43	3.57	1.00	0.965	2.25	1.00	0.965	1.11
×7/16	9.27	1.43	1.55	1.68	3.28	3.42	3.56	1.00	0.912	2.26	0.998	0.912	1.12
×3/8	8.00	1.42	1.54	1.67	3.26	3.40	3.54	1.00	0.840	2.27	0.928	0.840	1.12
2L6×6×1	22.0	2.58	2.72	2.86	2.58	2.72	2.86	1.00	1.00	1.79	1.00	1.00	1.79
×7/8	19.5	2.56	2.70	2.84	2.56	2.70	2.84	1.00	1.00	1.81	1.00	1.00	1.81
×3/4	16.9	2.54	2.67	2.81	2.54	2.67	2.81	1.00	1.00	1.82	1.00	1.00	1.82
×5/8	14.3	2.52	2.65	2.79	2.52	2.65	2.79	1.00	1.00	1.84	1.00	1.00	1.84
×9/16	12.9	2.51	2.64	2.78	2.51	2.64	2.78	1.00	1.00	1.85	1.00	1.00	1.85
×1/2	11.5	2.50	2.63	2.76	2.50	2.63	2.76	1.00	1.00	1.86	1.00	1.00	1.86
×7/16	10.2	2.49	2.62	2.75	2.49	2.62	2.75	1.00	0.973	1.86	1.00	0.973	1.86
×3/8	8.76	2.48	2.60	2.74	2.48	2.60	2.74	0.998	0.912	1.87	0.998	0.912	1.87
×5/16	7.34	2.47	2.59	2.72	2.47	2.59	2.72	0.914	0.826	1.88	0.914	0.826	1.88

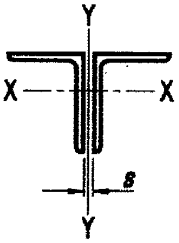
Note: For compactness criteria, refer to the end of Table 1–7

Table 1–15 (continued)
Double Angles
Properties

2L8–2L6

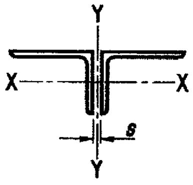
Shape	Flexural-Torsional Properties												Single Angle Properties	
	Long Legs Vertical						Short Legs Vertical							
	Back to Back of Angles, in.						Back to Back of Angles, in.						Area, A	r_z
	0		3/8		3/4		0		3/8		3/4			
	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	in.^{2}	in.
2L8×8×1 1/8	4.56	0.837	4.66	0.844	4.77	0.851	4.56	0.837	4.66	0.844	4.77	0.851	16.8	1.56
×1	4.56	0.834	4.66	0.841	4.77	0.848	4.56	0.834	4.66	0.841	4.77	0.848	15.1	1.56
×7/8	4.56	0.831	4.66	0.838	4.76	0.845	4.56	0.831	4.66	0.838	4.76	0.845	13.3	1.57
×3/4	4.56	0.829	4.66	0.836	4.76	0.843	4.56	0.829	4.66	0.836	4.76	0.843	11.5	1.57
×5/8	4.56	0.826	4.66	0.833	4.76	0.840	4.56	0.826	4.66	0.833	4.76	0.840	9.69	1.58
×9/16	4.56	0.825	4.65	0.832	4.75	0.839	4.56	0.825	4.65	0.832	4.75	0.839	8.77	1.58
×1/2	4.56	0.824	4.65	0.831	4.75	0.837	4.56	0.824	4.65	0.831	4.75	0.837	7.84	1.59
2L8×6×1	4.06	0.721	4.14	0.732	4.23	0.742	4.18	0.924	4.30	0.929	4.43	0.933	13.1	1.28
×7/8	4.07	0.718	4.14	0.728	4.23	0.739	4.17	0.922	4.29	0.926	4.42	0.930	11.5	1.28
×3/4	4.07	0.714	4.15	0.725	4.23	0.735	4.17	0.919	4.28	0.924	4.40	0.928	9.99	1.29
×5/8	4.08	0.712	4.16	0.722	4.24	0.732	4.16	0.917	4.27	0.921	4.39	0.926	8.41	1.29
×9/16	4.09	0.710	4.16	0.720	4.24	0.731	4.15	0.916	4.27	0.920	4.39	0.924	7.61	1.30
×1/2	4.09	0.709	4.16	0.719	4.24	0.729	4.15	0.915	4.26	0.919	4.38	0.923	6.80	1.30
×7/16	4.09	0.708	4.16	0.718	4.24	0.728	4.15	0.913	4.26	0.918	4.38	0.922	5.99	1.31
2L8×4×1	3.86	0.568	3.91	0.580	3.97	0.594	4.11	0.983	4.25	0.984	4.39	0.985	11.1	0.844
×7/8	3.87	0.566	3.92	0.577	3.98	0.590	4.09	0.981	4.22	0.982	4.37	0.984	9.79	0.846
×3/4	3.88	0.564	3.93	0.575	3.99	0.587	4.07	0.980	4.20	0.981	4.35	0.983	8.49	0.850
×5/8	3.89	0.562	3.94	0.573	3.99	0.585	4.05	0.979	4.18	0.980	4.32	0.981	7.16	0.856
×9/16	3.90	0.562	3.94	0.572	4.00	0.584	4.04	0.978	4.17	0.980	4.31	0.981	6.49	0.859
×1/2	3.90	0.561	3.95	0.571	4.00	0.583	4.03	0.978	4.16	0.979	4.30	0.980	5.80	0.863
×7/16	3.91	0.561	3.95	0.571	4.00	0.582	4.02	0.977	4.15	0.978	4.29	0.980	5.11	0.867
2L7×4×3/4	3.41	0.611	3.47	0.624	3.53	0.639	3.57	0.969	3.70	0.971	3.84	0.973	7.70	0.855
×5/8	3.42	0.608	3.47	0.621	3.54	0.635	3.55	0.967	3.68	0.969	3.82	0.971	6.50	0.860
×1/2	3.43	0.606	3.48	0.618	3.55	0.632	3.53	0.965	3.66	0.968	3.80	0.970	5.26	0.866
×7/16	3.43	0.605	3.49	0.617	3.55	0.630	3.53	0.964	3.66	0.967	3.79	0.969	4.63	0.869
×3/8	3.44	0.605	3.49	0.616	3.55	0.629	3.52	0.963	3.65	0.966	3.78	0.968	4.00	0.873
2L6×6×1	3.42	0.843	3.53	0.852	3.64	0.861	3.42	0.843	3.53	0.852	3.64	0.861	11.0	1.17
×7/8	3.42	0.839	3.53	0.848	3.63	0.857	3.42	0.839	3.53	0.848	3.63	0.857	9.75	1.17
×3/4	3.42	0.835	3.52	0.844	3.63	0.853	3.42	0.835	3.52	0.844	3.63	0.853	8.46	1.17
×5/8	3.42	0.831	3.52	0.840	3.62	0.849	3.42	0.831	3.52	0.840	3.62	0.849	7.13	1.17
×9/16	3.42	0.829	3.52	0.838	3.62	0.847	3.42	0.829	3.52	0.838	3.62	0.847	6.45	1.18
×1/2	3.42	0.827	3.52	0.836	3.62	0.846	3.42	0.827	3.52	0.836	3.62	0.846	5.77	1.18
×7/16	3.42	0.826	3.52	0.835	3.62	0.844	3.42	0.826	3.52	0.835	3.62	0.844	5.08	1.18
×3/8	3.42	0.824	3.51	0.833	3.61	0.842	3.42	0.824	3.51	0.833	3.61	0.842	4.38	1.19
×5/16	3.42	0.823	3.51	0.832	3.61	0.841	3.42	0.823	3.51	0.832	3.61	0.841	3.67	1.19

Note: For compactness criteria, refer to the end of Table 1–7



LLBB

Table 1-15 (continued)
Double Angles
Properties



SLBB

Shape	Area	Axis Y-Y						LLBB			SLBB		
		Radius of Gyration						Q_s		r_x	Q_s		r_x
		LLBB			SLBB			Angles in Contact	Angles Separated		Angles in Contact	Angles Separated	
		Separation, *s*, in.			Separation, *s*, in.								
	in.^{2}	0	3/8	3/4	0	3/8	3/4			in.			in.
2L6×4×7/8	16.0	1.57	1.71	1.86	2.82	2.96	3.11	1.00	1.00	1.86	1.00	1.00	1.10
×3/4	13.9	1.55	1.68	1.83	2.80	2.94	3.08	1.00	1.00	1.88	1.00	1.00	1.12
×5/8	11.7	1.53	1.66	1.80	2.77	2.91	3.06	1.00	1.00	1.89	1.00	1.00	1.13
×9/16	10.6	1.52	1.65	1.79	2.76	2.90	3.04	1.00	1.00	1.90	1.00	1.00	1.14
×1/2	9.50	1.51	1.64	1.77	2.75	2.89	3.03	1.00	1.00	1.91	1.00	1.00	1.14
×7/16	8.36	1.50	1.62	1.76	2.74	2.88	3.02	1.00	0.973	1.92	1.00	0.973	1.15
×3/8	7.22	1.49	1.61	1.75	2.73	2.86	3.00	1.00	0.912	1.93	0.998	0.912	1.16
×5/16	6.05	1.48	1.60	1.74	2.72	2.85	2.99	1.00	0.826	1.94	0.914	0.826	1.17
2L6×3½×1/2	9.04	1.27	1.40	1.54	2.82	2.96	3.11	1.00	1.00	1.92	1.00	1.00	0.968
×3/8	6.88	1.26	1.38	1.52	2.80	2.94	3.08	1.00	0.912	1.93	0.998	0.912	0.984
×5/16	5.78	1.25	1.37	1.50	2.78	2.92	3.06	1.00	0.826	1.94	0.914	0.826	0.991
2L5×5×7/8	16.0	2.16	2.30	2.44	2.16	2.30	2.44	1.00	1.00	1.49	1.00	1.00	1.49
×3/4	14.0	2.13	2.27	2.41	2.13	2.27	2.41	1.00	1.00	1.50	1.00	1.00	1.50
×5/8	11.8	2.11	2.25	2.39	2.11	2.25	2.39	1.00	1.00	1.52	1.00	1.00	1.52
×1/2	9.58	2.09	2.22	2.36	2.09	2.22	2.36	1.00	1.00	1.53	1.00	1.00	1.53
×7/16	8.44	2.08	2.21	2.35	2.08	2.21	2.35	1.00	1.00	1.54	1.00	1.00	1.54
×3/8	7.30	2.07	2.20	2.34	2.07	2.20	2.34	1.00	0.983	1.55	1.00	0.983	1.55
×5/16	6.13	2.06	2.19	2.32	2.06	2.19	2.32	0.998	0.912	1.56	0.998	0.912	1.56
2L5×3½×3/4	11.6	1.39	1.53	1.68	2.33	2.47	2.62	1.00	1.00	1.55	1.00	1.00	0.974
×5/8	9.85	1.37	1.50	1.65	2.30	2.45	2.59	1.00	1.00	1.56	1.00	1.00	0.987
×1/2	8.01	1.35	1.48	1.62	2.28	2.42	2.57	1.00	1.00	1.58	1.00	1.00	1.00
×3/8	6.10	1.33	1.46	1.59	2.26	2.39	2.54	1.00	0.983	1.59	1.00	0.983	1.02
×5/16	5.12	1.32	1.44	1.58	2.25	2.38	2.52	1.00	0.912	1.60	0.998	0.912	1.02
×1/4	4.13	1.31	1.43	1.57	2.23	2.37	2.51	1.00	0.804	1.61	0.894	0.804	1.03
2L5×3×1/2	7.51	1.11	1.24	1.39	2.35	2.50	2.64	1.00	1.00	1.58	1.00	1.00	0.824
×7/16	6.62	1.10	1.23	1.38	2.34	2.48	2.63	1.00	1.00	1.59	1.00	1.00	0.831
×3/8	5.73	1.09	1.22	1.36	2.33	2.47	2.62	1.00	0.983	1.60	1.00	0.983	0.838
×5/16	4.81	1.08	1.21	1.35	2.32	2.46	2.60	1.00	0.912	1.61	0.998	0.912	0.846
×1/4	3.88	1.07	1.19	1.33	2.30	2.44	2.58	1.00	0.804	1.62	0.894	0.804	0.853

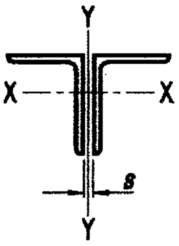
Note: For compactness criteria, refer to the end of Table 1–7

Table 1–15 (continued)
Double Angles
Properties

2L6–2L5

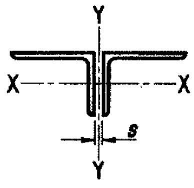
Shape	Flexural-Torsional Properties												Single Angle Properties	
	Long Legs Vertical						Short Legs Vertical							
	Back to Back of Angles, in.						Back to Back of Angles, in.						Area, A	r_z
	0		3/8		3/4		0		3/8		3/4			
	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	in.^{2}	in.
2L6×4×7/8	2.96	0.678	3.04	0.694	3.12	0.710	3.10	0.952	3.23	0.956	3.37	0.959	7.98	0.854
×3/4	2.97	0.673	3.04	0.688	3.12	0.705	3.09	0.949	3.22	0.953	3.35	0.957	6.94	0.856
×5/8	2.98	0.669	3.05	0.684	3.13	0.700	3.08	0.946	3.21	0.950	3.34	0.954	5.86	0.859
×9/16	2.98	0.667	3.05	0.682	3.13	0.697	3.07	0.945	3.20	0.949	3.33	0.953	5.31	0.861
×1/2	2.99	0.665	3.05	0.679	3.13	0.695	3.07	0.943	3.19	0.948	3.32	0.952	4.75	0.864
×7/16	2.99	0.663	3.06	0.678	3.13	0.693	3.06	0.942	3.19	0.946	3.31	0.950	4.18	0.867
×3/8	2.99	0.662	3.06	0.676	3.13	0.691	3.06	0.940	3.18	0.945	3.31	0.949	3.61	0.870
×5/16	3.00	0.661	3.06	0.674	3.13	0.689	3.05	0.939	3.17	0.944	3.30	0.948	3.03	0.874
2L6x3½×1/2	2.94	0.615	2.99	0.630	3.06	0.646	3.04	0.964	3.17	0.967	3.31	0.969	4.52	0.756
×3/8	2.95	0.613	3.00	0.627	3.07	0.642	3.02	0.962	3.15	0.965	3.29	0.967	3.44	0.763
×5/16	2.95	0.612	3.00	0.625	3.07	0.641	3.02	0.960	3.14	0.964	3.28	0.966	2.89	0.767
2L5×5×7/8	2.85	0.845	2.96	0.856	3.07	0.866	2.85	0.845	2.96	0.856	3.07	0.866	8.02	0.971
×3/4	2.85	0.840	2.95	0.851	3.06	0.861	2.85	0.840	2.95	0.851	3.06	0.861	6.98	0.972
×5/8	2.85	0.835	2.95	0.846	3.06	0.857	2.85	0.835	2.95	0.846	3.06	0.857	5.90	0.975
×1/2	2.85	0.830	2.94	0.842	3.05	0.852	2.85	0.830	2.94	0.842	3.05	0.852	4.79	0.980
×7/16	2.85	0.828	2.94	0.839	3.05	0.850	2.85	0.828	2.94	0.839	3.05	0.850	4.22	0.983
×3/8	2.84	0.826	2.94	0.838	3.04	0.848	2.84	0.826	2.94	0.838	3.04	0.848	3.65	0.986
×5/16	2.84	0.825	2.94	0.836	3.04	0.847	2.84	0.825	2.94	0.836	3.04	0.847	3.07	0.990
2L5x3½×3/4	2.49	0.699	2.57	0.717	2.66	0.736	2.60	0.943	2.73	0.949	2.86	0.953	5.82	0.744
×5/8	2.49	0.693	2.57	0.711	2.66	0.730	2.59	0.940	2.71	0.945	2.85	0.950	4.93	0.746
×1/2	2.50	0.688	2.58	0.705	2.66	0.724	2.58	0.936	2.70	0.942	2.83	0.947	4.00	0.750
×3/8	2.51	0.683	2.58	0.700	2.66	0.718	2.56	0.933	2.69	0.938	2.81	0.944	3.05	0.755
×5/16	2.51	0.682	2.58	0.698	2.66	0.716	2.56	0.931	2.68	0.937	2.81	0.942	2.56	0.758
×1/4	2.52	0.680	2.58	0.696	2.66	0.714	2.55	0.929	2.67	0.935	2.80	0.941	2.07	0.761
2L5×3×1/2	2.44	0.628	2.51	0.646	2.58	0.667	2.54	0.962	2.68	0.966	2.81	0.969	3.75	0.642
×7/16	2.45	0.626	2.51	0.644	2.58	0.664	2.54	0.961	2.67	0.964	2.80	0.968	3.31	0.644
×3/8	2.45	0.624	2.51	0.642	2.59	0.661	2.53	0.959	2.66	0.963	2.79	0.967	2.86	0.646
×5/16	2.46	0.623	2.52	0.640	2.59	0.659	2.52	0.958	2.65	0.962	2.78	0.965	2.41	0.649
×1/4	2.46	0.622	2.52	0.638	2.59	0.657	2.51	0.957	2.64	0.961	2.77	0.964	1.94	0.652

Note: For compactness criteria, refer to the end of Table 1–7



LLBB

Table 1–15 (continued)
Double Angles
Properties



SLBB

Shape	Area	Axis Y-Y Radius of Gyration LLBB Separation, s, in.			Axis Y-Y Radius of Gyration SLBB Separation, s, in.			LLBB Q_s Angles in Contact	LLBB Q_s Angles Separated	LLBB r_x	SLBB Q_s Angles in Contact	SLBB Q_s Angles Separated	SLBB r_x
	in.^{2}	0	3/8	3/4	0	3/8	3/4			in.			in.
2L4×4×3/4	10.9	1.73	1.88	2.03	1.73	1.88	2.03	1.00	1.00	1.18	1.00	1.00	1.18
×5/8	9.21	1.71	1.85	2.00	1.71	1.85	2.00	1.00	1.00	1.20	1.00	1.00	1.20
×1/2	7.49	1.69	1.83	1.97	1.69	1.83	1.97	1.00	1.00	1.21	1.00	1.00	1.21
×7/16	6.61	1.68	1.81	1.96	1.68	1.81	1.96	1.00	1.00	1.22	1.00	1.00	1.22
×3/8	5.71	1.67	1.80	1.94	1.67	1.80	1.94	1.00	1.00	1.23	1.00	1.00	1.23
×5/16	4.80	1.66	1.79	1.93	1.66	1.79	1.93	1.00	0.997	1.24	1.00	0.997	1.24
×1/4	3.87	1.65	1.78	1.91	1.65	1.78	1.91	0.998	0.912	1.25	0.998	0.912	1.25
2L4×3 1/2×1/2	7.01	1.44	1.57	1.72	1.75	1.89	2.03	1.00	1.00	1.23	1.00	1.00	1.04
×3/8	5.35	1.42	1.55	1.69	1.73	1.86	2.00	1.00	1.00	1.25	1.00	1.00	1.05
×5/16	4.50	1.40	1.53	1.68	1.72	1.85	1.99	1.00	0.997	1.25	1.00	0.997	1.06
×1/4	3.63	1.39	1.52	1.66	1.70	1.83	1.97	1.00	0.912	1.26	0.998	0.912	1.07
2L4×3×5/8	7.98	1.21	1.35	1.50	1.84	1.98	2.13	1.00	1.00	1.23	1.00	1.00	0.845
×1/2	6.51	1.19	1.32	1.47	1.81	1.95	2.10	1.00	1.00	1.24	1.00	1.00	0.858
×3/8	4.98	1.17	1.30	1.44	1.79	1.93	2.07	1.00	1.00	1.26	1.00	1.00	0.873
×5/16	4.19	1.16	1.29	1.43	1.78	1.91	2.06	1.00	0.997	1.27	1.00	0.997	0.880
×1/4	3.38	1.15	1.27	1.41	1.76	1.90	2.04	1.00	0.912	1.27	0.998	0.912	0.887
2L3 1/2×3 1/2×1/2	6.53	1.49	1.63	1.77	1.49	1.63	1.77	1.00	1.00	1.05	1.00	1.00	1.05
×7/16	5.77	1.48	1.61	1.76	1.48	1.61	1.76	1.00	1.00	1.06	1.00	1.00	1.06
×3/8	5.00	1.47	1.60	1.74	1.47	1.60	1.74	1.00	1.00	1.07	1.00	1.00	1.07
×5/16	4.21	1.46	1.59	1.73	1.46	1.59	1.73	1.00	1.00	1.08	1.00	1.00	1.08
×1/4	3.41	1.44	1.57	1.72	1.44	1.57	1.72	1.00	0.965	1.09	1.00	0.965	1.09
2L3 1/2×3×1/2	6.04	1.23	1.37	1.52	1.55	1.69	1.84	1.00	1.00	1.07	1.00	1.00	0.877
×7/16	5.34	1.22	1.36	1.51	1.54	1.67	1.82	1.00	1.00	1.08	1.00	1.00	0.885
×3/8	4.63	1.21	1.35	1.49	1.52	1.66	1.81	1.00	1.00	1.09	1.00	1.00	0.892
×5/16	3.91	1.20	1.33	1.48	1.51	1.65	1.79	1.00	1.00	1.09	1.00	1.00	0.900
×1/4	3.16	1.19	1.32	1.46	1.50	1.63	1.78	1.00	0.965	1.10	1.00	0.965	0.908
2L3 1/2×2 1/2×1/2	5.53	0.992	1.13	1.28	1.62	1.76	1.91	1.00	1.00	1.08	1.00	1.00	0.701
×3/8	4.25	0.970	1.11	1.25	1.59	1.73	1.88	1.00	1.00	1.10	1.00	1.00	0.716
×5/16	3.58	0.960	1.09	1.24	1.58	1.72	1.87	1.00	1.00	1.11	1.00	1.00	0.723
×1/4	2.90	0.950	1.08	1.22	1.57	1.70	1.85	1.00	0.965	1.12	1.00	0.965	0.731

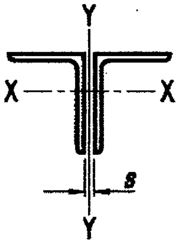
Note: For compactness criteria, refer to the end of Table 1–7

Table 1-15 (continued)
Double Angles
Properties

2L4–2L3½

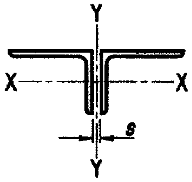
Shape	Flexural-Torsional Properties												Single Angle Properties	
	Long Legs Vertical						Short Legs Vertical							
	Back to Back of Angles, in.						Back to Back of Angles, in.						Area, *A*	r_z
	0		3/8		3/4		0		3/8		3/4			
	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	$\bar{r}_o$	*H*	in.²	in.
2L4×4×3/4	2.28	0.847	2.39	0.861	2.51	0.874	2.28	0.847	2.39	0.861	2.51	0.874	5.43	0.774
×5/8	2.28	0.841	2.39	0.854	2.50	0.868	2.28	0.841	2.39	0.854	2.50	0.868	4.61	0.774
×1/2	2.28	0.834	2.38	0.848	2.49	0.862	2.28	0.834	2.38	0.848	2.49	0.862	3.75	0.776
×7/16	2.28	0.832	2.38	0.846	2.49	0.859	2.28	0.832	2.38	0.846	2.49	0.859	3.30	0.777
×3/8	2.28	0.829	2.38	0.843	2.49	0.856	2.28	0.829	2.38	0.843	2.49	0.856	2.86	0.779
×5/16	2.28	0.826	2.37	0.840	2.48	0.854	2.28	0.826	2.37	0.840	2.48	0.854	2.40	0.781
×1/4	2.28	0.824	2.37	0.838	2.48	0.851	2.28	0.824	2.37	0.838	2.48	0.851	1.93	0.783
2L4×3½×1/2	2.14	0.784	2.23	0.802	2.33	0.819	2.16	0.882	2.28	0.893	2.40	0.904	3.50	0.716
×3/8	2.14	0.778	2.23	0.795	2.33	0.813	2.16	0.876	2.27	0.888	2.39	0.899	2.68	0.719
×5/16	2.14	0.775	2.23	0.792	2.33	0.810	2.16	0.874	2.26	0.885	2.38	0.896	2.25	0.721
×1/4	2.14	0.773	2.22	0.790	2.32	0.807	2.15	0.871	2.26	0.883	2.37	0.894	1.82	0.723
2L4×3×5/8	2.02	0.728	2.11	0.750	2.21	0.773	2.10	0.930	2.22	0.938	2.36	0.945	3.99	0.631
×1/2	2.02	0.721	2.11	0.743	2.20	0.765	2.09	0.925	2.21	0.933	2.34	0.940	3.25	0.633
×3/8	2.03	0.715	2.11	0.736	2.20	0.757	2.08	0.920	2.20	0.928	2.32	0.936	2.49	0.636
×5/16	2.03	0.712	2.11	0.733	2.20	0.754	2.07	0.918	2.19	0.926	2.32	0.934	2.09	0.638
×1/4	2.03	0.710	2.11	0.730	2.20	0.751	2.06	0.915	2.18	0.924	2.31	0.932	1.69	0.639
2L3½×3½×1/2	1.99	0.838	2.10	0.854	2.21	0.869	1.99	0.838	2.10	0.854	2.21	0.869	3.27	0.679
×7/16	1.99	0.835	2.09	0.851	2.21	0.866	1.99	0.835	2.09	0.851	2.21	0.866	2.89	0.681
×3/8	1.99	0.832	2.09	0.848	2.20	0.863	1.99	0.832	2.09	0.848	2.20	0.863	2.50	0.683
×5/16	1.99	0.829	2.09	0.845	2.20	0.860	1.99	0.829	2.09	0.845	2.20	0.860	2.10	0.685
×1/4	1.99	0.826	2.08	0.842	2.19	0.857	1.99	0.826	2.08	0.842	2.19	0.857	1.70	0.688
2L3½×3×1/2	1.85	0.780	1.94	0.801	2.05	0.822	1.88	0.892	2.00	0.904	2.13	0.915	3.02	0.618
×7/16	1.85	0.776	1.94	0.797	2.05	0.818	1.88	0.889	1.99	0.901	2.12	0.912	2.67	0.620
×3/8	1.85	0.773	1.94	0.794	2.05	0.814	1.88	0.885	1.99	0.898	2.11	0.910	2.32	0.622
×5/16	1.85	0.770	1.94	0.790	2.04	0.811	1.87	0.883	1.98	0.895	2.11	0.907	1.95	0.624
×1/4	1.85	0.767	1.94	0.787	2.04	0.807	1.87	0.880	1.98	0.893	2.10	0.905	1.58	0.628
2L3½×2½×1/2	1.75	0.706	1.83	0.732	1.93	0.759	1.82	0.938	1.95	0.946	2.08	0.953	2.76	0.532
×3/8	1.75	0.698	1.83	0.724	1.93	0.750	1.81	0.933	1.93	0.941	2.07	0.949	2.12	0.535
×5/16	1.76	0.695	1.83	0.720	1.92	0.746	1.80	0.930	1.92	0.939	2.06	0.947	1.79	0.538
×1/4	1.76	0.693	1.83	0.717	1.92	0.742	1.80	0.928	1.92	0.937	2.05	0.944	1.45	0.541

Note: For compactness criteria, refer to the end of Table 1-7



LLBB

Table 1–15 (continued)
Double Angles
Properties



SLBB

Shape	Area	Axis Y-Y						LLBB			SLBB		
		Radius of Gyration						Q_s		r_x	Q_s		r_x
		LLBB			SLBB			Angles in Contact	Angles Separated		Angles in Contact	Angles Separated	
		Separation, *s*, in.			Separation, *s*, in.								
	in.^{2}	0	3/8	3/4	0	3/8	3/4			in.			in.
2L3×3×1/2	5.50	1.29	1.43	1.58	1.29	1.43	1.58	1.00	1.00	0.895	1.00	1.00	0.895
×7/16	4.86	1.28	1.42	1.57	1.28	1.42	1.57	1.00	1.00	0.903	1.00	1.00	0.903
×3/8	4.22	1.27	1.41	1.55	1.27	1.41	1.55	1.00	1.00	0.910	1.00	1.00	0.910
×5/16	3.55	1.26	1.39	1.54	1.26	1.39	1.54	1.00	1.00	0.918	1.00	1.00	0.918
×1/4	2.87	1.25	1.38	1.52	1.25	1.38	1.52	1.00	1.00	0.926	1.00	1.00	0.926
×3/16	2.18	1.24	1.37	1.51	1.24	1.37	1.51	0.998	0.912	0.933	0.998	0.912	0.933
2L3×2 1/2×1/2	5.01	1.04	1.18	1.33	1.35	1.49	1.64	1.00	1.00	0.910	1.00	1.00	0.718
×7/16	4.44	1.02	1.16	1.32	1.34	1.48	1.63	1.00	1.00	0.917	1.00	1.00	0.724
×3/8	3.86	1.01	1.15	1.30	1.32	1.46	1.61	1.00	1.00	0.924	1.00	1.00	0.731
×5/16	3.25	1.00	1.14	1.29	1.31	1.45	1.60	1.00	1.00	0.932	1.00	1.00	0.739
×1/4	2.64	0.991	1.12	1.27	1.30	1.44	1.58	1.00	1.00	0.940	1.00	1.00	0.746
×3/16	2.00	0.980	1.11	1.25	1.29	1.42	1.57	1.00	0.912	0.947	0.998	0.912	0.753
2L3×2×1/2	4.53	0.795	0.940	1.10	1.42	1.56	1.72	1.00	1.00	0.922	1.00	1.00	0.543
×3/8	3.50	0.771	0.911	1.07	1.39	1.54	1.69	1.00	1.00	0.937	1.00	1.00	0.555
×5/16	2.96	0.760	0.897	1.05	1.38	1.52	1.67	1.00	1.00	0.945	1.00	1.00	0.562
×1/4	2.40	0.749	0.883	1.03	1.37	1.51	1.66	1.00	1.00	0.953	1.00	1.00	0.569
×3/16	1.83	0.739	0.869	1.02	1.35	1.49	1.64	1.00	0.912	0.961	0.998	0.912	0.577
2L2 1/2×2 1/2×1/2	4.50	1.09	1.23	1.39	1.09	1.23	1.39	1.00	1.00	0.735	1.00	1.00	0.735
×3/8	3.47	1.07	1.21	1.36	1.07	1.21	1.36	1.00	1.00	0.749	1.00	1.00	0.749
×5/16	2.93	1.05	1.19	1.34	1.05	1.19	1.34	1.00	1.00	0.756	1.00	1.00	0.756
×1/4	2.37	1.04	1.18	1.33	1.04	1.18	1.33	1.00	1.00	0.764	1.00	1.00	0.764
×3/16	1.80	1.03	1.17	1.31	1.03	1.17	1.31	1.00	0.983	0.771	1.00	0.983	0.771
2L2 1/2×2×3/8	3.11	0.815	0.957	1.11	1.13	1.27	1.42	1.00	1.00	0.766	1.00	1.00	0.574
×5/16	2.64	0.804	0.943	1.10	1.12	1.26	1.41	1.00	1.00	0.774	1.00	1.00	0.581
×1/4	2.14	0.794	0.930	1.08	1.10	1.24	1.39	1.00	1.00	0.782	1.00	1.00	0.589
×3/16	1.64	0.784	0.916	1.07	1.09	1.23	1.38	1.00	0.983	0.790	1.00	0.983	0.597
2L2 1/2×1 1/2×1/4	1.89	0.554	0.694	0.852	1.17	1.32	1.47	1.00	1.00	0.792	1.00	1.00	0.411
×3/16	1.45	0.543	0.679	0.834	1.16	1.30	1.45	1.00	0.983	0.801	1.00	0.983	0.418
2L2×2×3/8	2.73	0.865	1.01	1.17	0.865	1.01	1.17	1.00	1.00	0.591	1.00	1.00	0.591
×5/16	2.32	0.853	0.996	1.15	0.853	0.996	1.15	1.00	1.00	0.598	1.00	1.00	0.598
×1/4	1.89	0.842	0.982	1.14	0.842	0.982	1.14	1.00	1.00	0.605	1.00	1.00	0.605
×3/16	1.44	0.831	0.967	1.12	0.831	0.967	1.12	1.00	1.00	0.612	1.00	1.00	0.612
×1/8	0.982	0.818	0.951	1.10	0.818	0.951	1.10	0.998	0.912	0.620	0.998	0.912	0.620

Note: For compactness criteria, refer to the end of Table 1–7

Table 1-15 (continued)
Double Angles
Properties

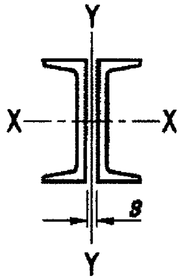
2L3–2L2

Shape	Flexural-Torsional Properties												Single Angle Properties	
	Long Legs Vertical						Short Legs Vertical							
	Back to Back of Angles, in.						Back to Back of Angles, in.						Area, A	r_z
	0		3/8		3/4		0		3/8		3/4			
	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	$\bar{r}_o$	H	in.^{2}	in.
2L3×3×1/2	1.71	0.842	1.82	0.861	1.94	0.878	1.71	0.842	1.82	0.861	1.94	0.878	2.75	0.580
×7/16	1.71	0.838	1.82	0.857	1.94	0.874	1.71	0.838	1.82	0.857	1.94	0.874	2.43	0.580
×3/8	1.71	0.834	1.81	0.853	1.93	0.870	1.71	0.834	1.81	0.853	1.93	0.870	2.11	0.581
×5/16	1.71	0.830	1.81	0.849	1.93	0.866	1.71	0.830	1.81	0.849	1.93	0.866	1.78	0.583
×1/4	1.71	0.827	1.81	0.845	1.92	0.863	1.71	0.827	1.81	0.845	1.92	0.863	1.44	0.585
×3/16	1.71	0.823	1.80	0.842	1.91	0.859	1.71	0.823	1.80	0.842	1.91	0.859	1.09	0.586
2L3×2 1/2×1/2	1.57	0.774	1.66	0.800	1.78	0.824	1.61	0.905	1.73	0.918	1.86	0.929	2.51	0.516
×7/16	1.57	0.769	1.66	0.795	1.77	0.819	1.60	0.901	1.72	0.914	1.85	0.926	2.22	0.516
×3/8	1.57	0.764	1.66	0.790	1.77	0.815	1.60	0.897	1.72	0.911	1.85	0.923	1.93	0.517
×5/16	1.57	0.760	1.66	0.785	1.76	0.810	1.59	0.893	1.71	0.907	1.84	0.920	1.63	0.518
×1/4	1.57	0.756	1.66	0.781	1.76	0.806	1.59	0.890	1.70	0.904	1.83	0.917	1.32	0.520
×3/16	1.57	0.753	1.65	0.778	1.75	0.802	1.58	0.887	1.70	0.901	1.82	0.914	1.00	0.521
2L3×2×1/2	1.47	0.684	1.55	0.717	1.66	0.751	1.55	0.955	1.69	0.962	1.83	0.968	2.26	0.425
×3/8	1.48	0.675	1.55	0.707	1.65	0.739	1.54	0.949	1.67	0.957	1.81	0.963	1.75	0.426
×5/16	1.48	0.671	1.56	0.702	1.65	0.734	1.53	0.946	1.66	0.954	1.80	0.961	1.48	0.428
×1/4	1.48	0.668	1.56	0.698	1.65	0.730	1.52	0.944	1.65	0.952	1.79	0.959	1.20	0.431
×3/16	1.49	0.666	1.55	0.695	1.64	0.726	1.52	0.941	1.64	0.950	1.78	0.957	0.917	0.435
2L2 1/2×2 1/2×1/2	1.43	0.850	1.54	0.871	1.67	0.890	1.43	0.850	1.54	0.871	1.67	0.890	2.25	0.481
×3/8	1.42	0.839	1.53	0.861	1.65	0.881	1.42	0.839	1.53	0.861	1.65	0.881	1.73	0.481
×5/16	1.42	0.834	1.53	0.856	1.65	0.876	1.42	0.834	1.53	0.856	1.65	0.876	1.46	0.481
×1/4	1.42	0.829	1.52	0.852	1.64	0.872	1.42	0.829	1.52	0.852	1.64	0.872	1.19	0.482
×3/16	1.42	0.825	1.52	0.847	1.63	0.868	1.42	0.825	1.52	0.847	1.63	0.868	0.901	0.482
2L2 1/2×2×3/8	1.29	0.754	1.38	0.786	1.49	0.817	1.32	0.913	1.45	0.927	1.59	0.939	1.56	0.419
×5/16	1.29	0.748	1.38	0.781	1.49	0.812	1.32	0.909	1.44	0.923	1.58	0.936	1.32	0.420
×1/4	1.29	0.744	1.38	0.775	1.49	0.806	1.32	0.904	1.43	0.920	1.57	0.933	1.07	0.423
×3/16	1.29	0.740	1.38	0.771	1.48	0.801	1.31	0.901	1.43	0.916	1.56	0.929	0.818	0.426
2L2 1/2×1 1/2×1/4	1.22	0.630	1.29	0.669	1.38	0.712	1.27	0.962	1.40	0.969	1.55	0.975	0.947	0.321
×3/16	1.22	0.627	1.29	0.665	1.38	0.706	1.26	0.959	1.39	0.967	1.53	0.973	0.724	0.324
2L2×2×3/8	1.14	0.847	1.25	0.874	1.38	0.897	1.14	0.847	1.25	0.874	1.38	0.897	1.37	0.386
×5/16	1.14	0.841	1.25	0.868	1.37	0.891	1.14	0.841	1.25	0.868	1.37	0.891	1.16	0.386
×1/4	1.13	0.835	1.24	0.862	1.37	0.886	1.13	0.835	1.24	0.862	1.37	0.886	0.944	0.387
×3/16	1.13	0.830	1.24	0.857	1.36	0.882	1.13	0.830	1.24	0.857	1.36	0.882	0.722	0.389
×1/8	1.13	0.826	1.23	0.853	1.35	0.877	1.13	0.826	1.23	0.853	1.35	0.877	0.491	0.391

Note: For compactness criteria, refer to the end of Table 1-7

2C SHAPES

Table 1–16
2C Shapes
Properties



Shape	Area, A	Axis Y-Y												Axis X-X
		Separation, s, in.												
		0				3/8				3/4				r_x
		I	S	r	Z	I	S	r	Z	I	S	r	Z	
	in.^{2}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.
2C15×50	29.4	40.7	11.0	1.18	30.7	50.5	12.9	1.31	36.2	62.4	15.3	1.46	41.7	5.24
×40	23.5	32.6	9.25	1.18	22.9	40.2	10.9	1.31	27.3	49.6	12.7	1.45	31.7	5.44
×33.9	19.9	28.5	8.38	1.20	19.0	35.1	9.78	1.33	22.7	43.1	11.4	1.47	26.4	5.63
2C12×30	17.6	18.2	5.75	1.02	15.1	23.3	6.94	1.15	18.4	29.6	8.36	1.30	21.7	4.29
×25	14.7	15.6	5.11	1.03	12.1	19.8	6.12	1.16	14.9	25.0	7.32	1.31	17.6	4.43
×20.7	12.2	13.6	4.64	1.06	10.0	17.2	5.51	1.19	12.3	21.7	6.55	1.34	14.6	4.61
2C10×30	17.6	15.3	5.04	0.931	15.3	20.2	6.27	1.07	18.6	26.3	7.73	1.22	21.9	3.42
×25	14.7	12.3	4.25	0.914	11.8	16.2	5.27	1.05	14.5	21.1	6.48	1.20	17.3	3.52
×20	11.7	9.91	3.62	0.918	8.84	13.0	4.44	1.05	11.0	16.9	5.43	1.20	13.2	3.66
×15.3	8.96	8.14	3.13	0.953	6.69	10.6	3.80	1.09	8.37	13.7	4.59	1.23	10.0	3.87
2C9×20	11.7	8.80	3.32	0.866	8.76	11.8	4.15	1.00	11.0	15.6	5.15	1.15	13.2	3.22
×15	8.81	6.86	2.76	0.882	6.25	9.10	3.41	1.02	7.90	12.0	4.19	1.17	9.55	3.40
×13.4	7.88	6.34	2.61	0.897	5.59	8.39	3.20	1.03	7.07	11.0	3.92	1.18	8.55	3.49
2C8×18.7	11.0	7.46	2.95	0.823	8.12	10.2	3.75	0.962	10.2	13.7	4.71	1.11	12.3	2.82
×13.7	8.07	5.51	2.35	0.826	5.49	7.47	2.95	0.962	7.00	10.0	3.68	1.11	8.52	2.99
×11.5	6.74	4.82	2.13	0.846	4.57	6.50	2.66	0.982	5.83	8.66	3.29	1.13	7.10	3.11
2C7×14.7	8.66	5.18	2.25	0.773	5.94	7.21	2.90	0.912	7.57	9.85	3.68	1.07	9.19	2.51
×12.2	7.19	4.30	1.96	0.773	4.69	5.97	2.51	0.911	6.04	8.14	3.17	1.06	7.39	2.60
×9.8	5.73	3.59	1.72	0.791	3.69	4.95	2.17	0.929	4.76	6.72	2.73	1.08	5.84	2.72
2C6×13	7.63	4.11	1.91	0.734	5.13	5.85	2.50	0.876	6.56	8.13	3.21	1.03	7.99	2.13
×10.5	6.15	3.26	1.60	0.728	3.86	4.63	2.08	0.867	5.02	6.43	2.67	1.02	6.17	2.22
×8.2	4.78	2.63	1.37	0.741	2.93	3.72	1.76	0.881	3.82	5.14	2.24	1.04	4.72	2.34
2C5×9	5.28	2.45	1.30	0.682	3.22	3.59	1.73	0.824	4.21	5.09	2.25	0.982	5.20	1.83
×6.7	3.93	1.86	1.06	0.688	2.36	2.71	1.40	0.831	3.09	3.84	1.81	0.989	3.83	1.95
2C4×7.2	4.26	1.75	1.02	0.641	2.52	2.63	1.38	0.786	3.32	3.81	1.82	0.946	4.12	1.47
×5.4	3.16	1.29	0.812	0.637	1.86	1.94	1.10	0.783	2.45	2.82	1.44	0.943	3.05	1.56
×4.5	2.76	1.25	0.789	0.673	1.95	1.86	1.05	0.820	2.47	2.66	1.36	0.981	2.98	1.63
2C3×6	3.52	1.33	0.833	0.614	2.12	2.06	1.15	0.764	2.78	3.03	1.54	0.927	3.44	1.08
×5	2.94	1.05	0.699	0.597	1.65	1.63	0.969	0.746	2.20	2.43	1.30	0.909	2.75	1.12
×4.1	2.41	0.842	0.597	0.591	1.43	1.32	0.827	0.741	1.88	1.97	1.10	0.905	2.33	1.17
×3.5	2.18	0.766	0.558	0.593	1.37	1.20	0.772	0.743	1.78	1.80	1.03	0.908	2.19	1.20

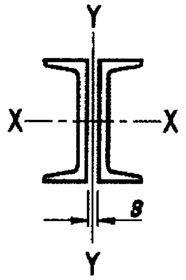
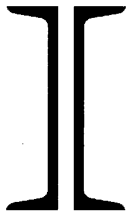


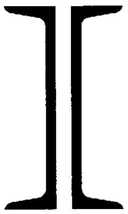
Table 1–17
2MC Shapes
Properties



2MC18–2MC6

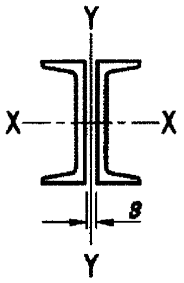
Shape	Area, *A*	Axis Y-Y												Axis X-X
		Separation, *s*, in.												
		0				3/8				3/4				r_x
		I	*S*	*r*	*Z*	*I*	*S*	*r*	*Z*	*I*	*S*	*r*	*Z*	
	in.^{2}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.
2MC18×58	34.1	60.6	14.4	1.33	37.5	72.8	16.6	1.46	43.9	87.5	19.1	1.60	50.3	6.29
×51.9	30.5	55.0	13.4	1.34	32.7	65.9	15.4	1.47	38.4	79.0	17.6	1.61	44.1	6.40
×45.8	26.9	50.1	12.5	1.36	28.3	59.8	14.3	1.49	33.4	71.4	16.3	1.63	38.4	6.56
×42.7	25.1	47.8	12.1	1.38	26.4	57.0	13.8	1.51	31.1	67.9	15.7	1.64	35.8	6.65
2MC13×50	29.4	60.7	13.8	1.44	37.0	72.5	15.8	1.57	42.5	86.3	18.0	1.71	48.0	4.62
×40	23.5	49.1	11.7	1.45	28.0	58.4	13.4	1.58	32.4	69.4	15.2	1.72	36.8	4.82
×35	20.6	44.3	10.9	1.47	24.2	52.6	12.3	1.60	28.1	62.3	14.0	1.74	31.9	4.95
×31.8	18.7	41.5	10.4	1.49	22.1	49.2	11.7	1.62	25.6	58.2	13.3	1.76	29.1	5.06
2MC12×50	29.4	67.2	16.2	1.51	39.9	79.8	18.5	1.65	45.4	94.5	20.9	1.79	50.9	4.28
×45	26.4	59.9	14.9	1.51	34.6	71.1	16.9	1.64	39.6	84.1	19.2	1.79	44.5	4.36
×40	23.5	53.7	13.8	1.51	30.1	63.7	15.6	1.65	34.6	75.3	17.7	1.79	39.0	4.46
×35	20.5	48.0	12.7	1.53	26.0	56.8	14.4	1.66	29.9	67.1	16.2	1.81	33.7	4.59
×31	18.2	44.0	12.0	1.55	24.1	52.1	13.5	1.69	27.5	61.4	15.2	1.83	30.9	4.71
×10.6[c]	6.20	1.21	0.804	0.441	2.07	2.05	1.21	0.575	3.23	3.33	1.78	0.733	4.40	4.23
2MC10×41.1	24.2	60.0	13.9	1.58	33.6	70.7	15.7	1.71	38.1	83.1	17.7	1.85	42.6	3.60
×33.6	19.7	49.5	12.1	1.58	26.3	58.2	13.6	1.72	30.0	68.3	15.3	1.86	33.7	3.75
×28.5	16.7	43.5	11.0	1.61	22.2	51.1	12.3	1.75	25.3	59.8	13.8	1.89	28.5	3.88
2MC10×25	14.7	27.8	8.18	1.38	16.7	33.6	9.36	1.51	19.5	40.4	10.7	1.66	22.2	3.87
×22	12.9	25.4	7.67	1.40	16.6	30.7	8.76	1.54	19.0	36.8	10.0	1.69	21.4	3.98
2MC10×8.4[c]	4.91	1.05	0.700	0.462	1.70	1.75	1.03	0.596	2.62	2.79	1.49	0.753	3.54	3.61
×6.5[c]	3.90	0.414	0.354	0.326	0.947	0.835	0.615	0.463	1.68	1.53	0.990	0.626	2.41	3.43
2MC9×25.4	14.9	29.2	8.34	1.40	17.6	35.2	9.53	1.53	20.4	42.2	10.9	1.68	23.2	3.43
×23.9	14.0	27.8	8.05	1.41	16.5	33.4	9.19	1.54	19.1	40.1	10.5	1.69	21.8	3.48
2MC8×22.8	13.4	27.7	7.91	1.44	16.3	33.2	9.01	1.58	18.9	39.7	10.2	1.72	21.4	3.09
×21.4	12.6	26.3	7.63	1.45	16.0	31.6	8.68	1.59	18.4	37.7	9.86	1.73	20.7	3.13
2MC8×20	11.8	17.1	5.66	1.21	12.0	21.2	6.61	1.34	14.2	26.2	7.70	1.49	16.4	3.05
×18.7	11.0	16.2	5.45	1.21	11.2	20.1	6.35	1.35	13.3	24.8	7.39	1.50	15.4	3.09
2MC8×8.5	5.00	2.16	1.15	0.658	2.53	3.14	1.52	0.793	3.47	4.47	1.99	0.946	4.40	3.05
2MC7×22.7	13.3	29.0	8.06	1.47	17.1	34.7	9.16	1.61	19.6	41.3	10.4	1.76	22.1	2.67
×19.1	11.2	25.1	7.27	1.50	16.2	30.0	8.25	1.64	18.3	35.7	9.34	1.78	20.4	2.77
2MC6×18	10.6	25.0	7.13	1.54	16.2	29.8	8.07	1.68	18.2	35.3	9.11	1.83	20.1	2.37
×15.3	8.97	19.7	5.63	1.48	12.3	23.6	6.39	1.62	14.0	28.1	7.24	1.77	15.6	2.38

[c] Shape is slender for compression with F_y = 36 ksi.



2MC6–2MC3

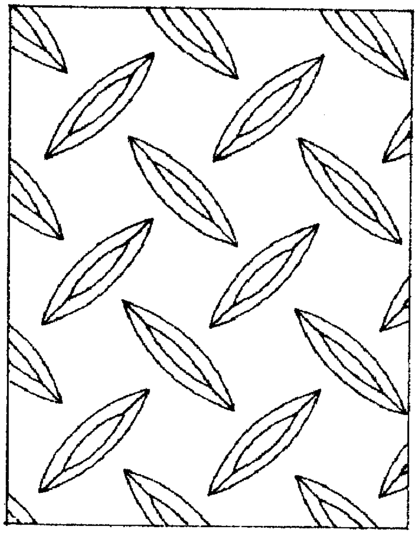
Table 1-17 (continued)
2MC Shapes
Properties

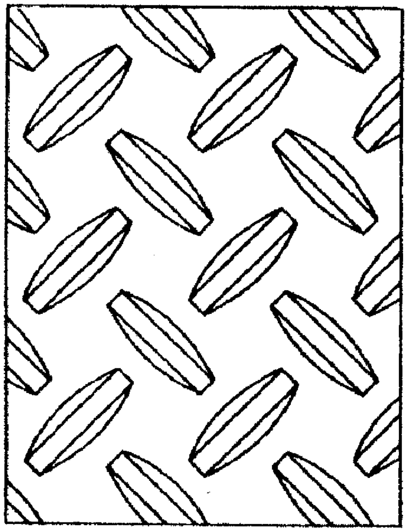


Shape	Area, A	Axis Y-Y												Axis X-X
		Separation, s, in.												
		0				3/8				3/4				r_x
		I	S	r	Z	I	S	r	Z	I	S	r	Z	
	in.^{2}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.^{4}	in.^{3}	in.	in.^{3}	in.
2MC6×16.3	9.58	15.8	5.26	1.28	11.4	19.4	6.10	1.42	13.2	23.8	7.05	1.58	15.0	2.33
×15.1	8.88	14.8	5.02	1.29	11.4	18.2	5.82	1.43	13.1	22.3	6.71	1.58	14.7	2.37
2MC6×12	7.06	7.21	2.89	1.01	6.01	9.32	3.47	1.15	7.34	11.9	4.15	1.30	8.66	2.30
2MC6×7	4.18	2.25	1.20	0.734	2.46	3.19	1.55	0.873	3.24	4.41	1.96	1.03	4.03	2.34
×6.5	3.89	2.15	1.16	0.744	2.42	3.04	1.49	0.883	3.15	4.20	1.89	1.04	3.88	2.38
2MC4×13.8	8.06	10.1	4.03	1.12	8.90	12.9	4.81	1.27	10.4	16.3	5.68	1.42	11.9	1.48
2MC3×7.1	4.22	3.13	1.62	0.862	3.74	4.31	2.03	1.01	4.53	5.79	2.50	1.17	5.32	1.14

Table 1–18

Weights of Raised-Pattern Floor Plates





Gauge No.	Wt., lb/ft^2	Nominal Thickness, in.	Wt., lb/ft^2	Nominal Thickness, in.	Wt., lb/ft^2
18	2.40	1/8	6.16	1/2	21.5
16	3.00	3/16	8.71	9/16	24.0
14	3.75	1/4	11.3	5/8	26.6
13	4.50	5/16	13.8	3/4	31.7
12	5.25	3/8	16.4	7/8	36.8
		7/16	18.9	1	41.9

Note: Thickness is measured near the edge of the plate, exclusive of raised pattern.

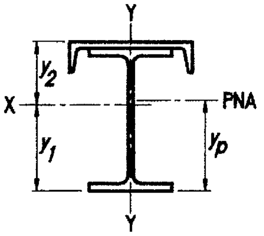
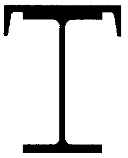


Table 1–19
W Shapes with Cap Channels
Properties

W-Shape	Channel	Total Wt.	Total Area	Axis X-X			
				I	$S_1 = \frac{I}{y_1}$	$S_2 = \frac{I}{y_2}$	r
		lb/ft	in.^{2}	in.^{4}	in.^{3}	in.^{3}	in.
W36×150	MC18×42.7	193	56.8	12000	553	831	14.6
	C15×33.9	184	54.2	11500	546	764	14.6
W33×141	MC18×42.7	184	54.1	10000	490	750	13.6
	C15×33.9	175	51.5	9580	484	689	13.6
W33×118	MC18×42.7	161	47.2	8280	400	656	13.2
	C15×33.9	152	44.6	7900	395	596	13.3
W30×116	MC18×42.7	159	46.8	6900	365	598	12.1
	C15×33.9	150	44.1	6590	360	544	12.2
W30×99	MC18×42.7	142	41.6	5830	304	533	11.8
	C15×33.9	133	39.0	5550	300	481	11.9
W27×94	C15×33.9	128	37.6	4530	268	435	11.0
W27×84	C15×33.9	118	34.7	4050	237	403	10.8
W24×84	C15×33.9	118	34.7	3340	217	367	9.82
	C12×20.7	105	30.8	3030	211	302	9.92
W24×68	C15×33.9	102	30.0	2710	173	321	9.51
	C12×20.7	88.7	26.1	2440	168	258	9.67
W21×68	C15×33.9	102	30.0	2180	156	287	8.52
	C12×20.7	88.7	26.1	1970	152	232	8.67
W21×62	C15×33.9	95.9	28.2	2000	142	272	8.41
	C12×20.7	82.7	24.3	1800	138	218	8.59
W18×50	C15×33.9	83.9	24.6	1250	100	211	7.12
	C12×20.7	70.7	20.7	1120	97.3	166	7.35
W16×36	C15×33.9	69.9	20.5	748	64.5	160	6.04
	C12×20.7	56.7	16.6	670	62.8	123	6.34
W14×30	C12×20.7	50.7	14.9	447	46.7	98.1	5.47
	C10×15.3	45.3	13.3	420	46.0	84.5	5.61
W12×26	C12×20.7	46.7	13.7	318	36.8	82.1	4.81
	C10×15.3	41.3	12.1	299	36.3	70.5	4.96

Note: Compactness criteria not addressed in this table.

Table 1–19 (continued)
W Shapes with Cap Channels
Properties



W-Shape	Channel	Axis X-X				Axis Y-Y			
		y_1	y_2	Z	y_p	I	S	r	Z
		in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}
W36×150	MC18×42.7	21.8	14.5	738	28.0	824	91.5	3.81	146
	C15×33.9	21.1	15.1	716	25.9	584	77.9	3.28	122
W33×141	MC18×42.7	20.4	13.3	652	27.0	800	88.9	3.85	142
	C15×33.9	19.8	13.9	635	24.9	561	74.8	3.30	118
W33×118	MC18×42.7	20.7	12.6	544	27.8	741	82.3	3.96	126
	C15×33.9	20.0	13.3	529	25.5	502	66.9	3.35	102
W30×116	MC18×42.7	18.9	11.5	492	26.1	718	79.8	3.92	124
	C15×33.9	18.3	12.1	480	23.8	479	63.8	3.29	100
W30×99	MC18×42.7	19.2	10.9	412	26.4	682	75.8	4.05	114
	C15×33.9	18.5	11.5	408	24.4	442	59.0	3.37	89.4
W27×94	C15×33.9	16.9	10.4	357	23.6	439	58.5	3.41	89.6
W27×84	C15×33.9	17.1	10.0	316	23.9	420	56.0	3.48	83.9
W24×84	C15×33.9	15.4	9.10	286	21.6	409	54.5	3.43	83.4
	C12×20.7	14.3	10.0	275	18.5	223	37.2	2.69	58.2
W24×68	C15×33.9	15.7	8.46	232	21.7	385	51.3	3.58	75.3
	C12×20.7	14.5	9.49	224	19.2	199	33.2	2.76	50.1
W21×68	C15×33.9	13.9	7.59	207	19.3	379	50.6	3.56	75.1
	C12×20.7	12.9	8.49	200	17.6	194	32.3	2.72	50.0
W21×62	C15×33.9	14.1	7.33	189	19.4	372	49.6	3.63	72.5
	C12×20.7	13.0	8.26	183	18.1	186	31.1	2.77	47.3
W18×50	C15×33.9	12.5	5.92	133	16.9	354	47.3	3.79	67.3
	C12×20.7	11.5	6.76	127	16.1	169	28.2	2.85	42.2
W16×36	C15×33.9	11.6	4.67	86.8	15.2	339	45.2	4.06	61.6
	C12×20.7	10.7	5.47	83.2	14.6	153	25.6	3.04	36.4
W14×30	C12×20.7	9.57	4.55	62.0	12.9	149	24.8	3.16	34.6
	C10×15.3	9.11	4.97	60.3	12.6	86.8	17.4	2.55	24.9
W12×26	C12×20.7	8.63	3.87	48.2	11.6	146	24.4	3.27	33.7
	C10×15.3	8.22	4.24	47.0	11.3	84.5	16.9	2.64	24.1

Note: Compactness criteria not addressed in this table.

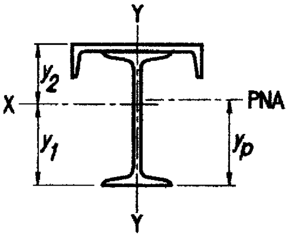


Table 1–20

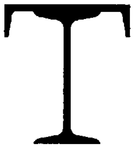
S Shapes with Cap Channels

Properties

S-Shape	Channel	Total Wt.	Total Area	Axis X-X			
				I	$S_1 = \frac{I}{y_1}$	$S_2 = \frac{I}{y_2}$	r
		lb/ft	in.^{2}	in.^{4}	in.^{3}	in.^{3}	in.
S24×80	C12×20.7	101	29.5	2750	191	278	9.66
	C10×15.3	95.3	27.9	2610	188	252	9.67
S20×66	C12×20.7	86.7	25.5	1620	132	202	7.97
	C10×15.3	81.3	23.9	1530	129	181	8.00
S15×42.9	C10×15.3	58.2	17.1	615	65.7	105	6.00
	C8×11.5	54.4	16.0	583	64.7	93.9	6.04
S12×31.8	C10×15.3	47.1	13.8	314	40.2	71.2	4.77
	C8×11.5	43.3	12.7	297	39.6	63.0	4.84
S10×25.4	C10×15.3	40.7	11.9	185	27.5	52.7	3.94
	C8×11.5	36.9	10.8	175	27.1	46.3	4.02

Note: Compactness criteria not addressed in this table.

Table 1–20 (continued)
S Shapes with Cap Channels
Properties



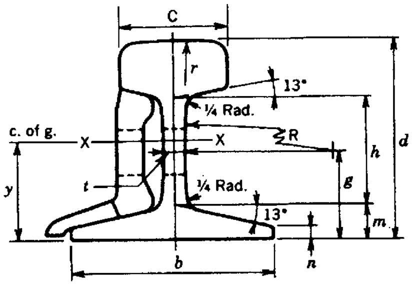
S-Shape	Channel	Axis X-X				Axis Y-Y			
		y_1	y_2	Z	y_p	I	S	r	Z
		in.	in.	in.^{3}	in.	in.^{4}	in.^{3}	in.	in.^{3}
S24×80	C12×20.7	14.4	9.90	256	18.1	171	28.5	2.41	46.4
	C10×15.3	13.9	10.4	246	16.5	109	21.8	1.98	36.8
S20×66	C12×20.7	12.3	7.99	180	16.0	156	26.1	2.48	41.0
	C10×15.3	11.8	8.44	173	14.4	94.7	18.9	1.99	31.3
S15×42.9	C10×15.3	9.37	5.87	87.6	12.8	81.5	16.3	2.18	25.0
	C8×11.5	9.01	6.21	86.5	11.6	46.8	11.7	1.71	18.7
S12×31.8	C10×15.3	7.82	4.42	54.0	10.6	76.5	15.3	2.36	22.3
	C8×11.5	7.50	4.72	52.4	10.3	41.8	10.5	1.82	16.1
S10×25.4	C10×15.3	6.73	3.51	37.2	9.03	73.9	14.8	2.49	20.9
	C8×11.5	6.45	3.77	36.1	8.82	39.2	9.81	1.90	14.6

Note: Compactness criteria not addressed in this table.

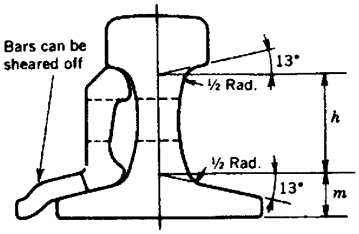
Table 1–21

Crane Rails

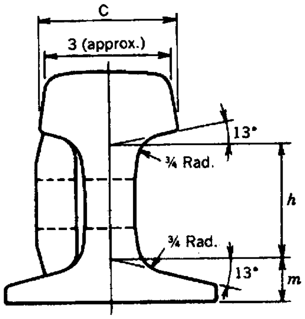
Dimensions and Properties



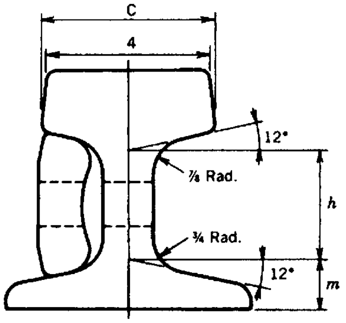
ASCE CRANE RAILS



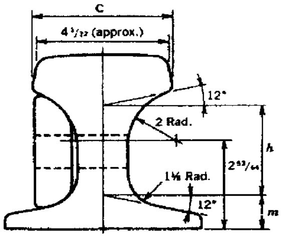
ASTM PROFILE 104



ASTM PROFILE 135



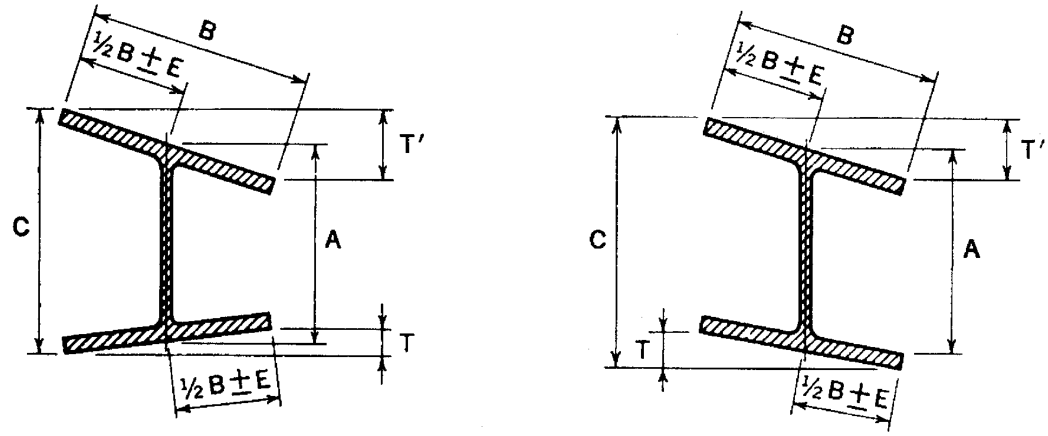
ASTM PROFILE 171



ASTM PROFILE 175

TYPE	Classification	Wt.	Depth, *d*	Gage, *g*	Base			Head		Web			Axis X-X				
															S		
					b	*m*	*n*	*c*	*r*	*t*	*h*	*R*	Area	*I*	Head	Base	*y*
		lb/yd	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.	in.^{2}	in.^{4}	in.^{3}	in.^{3}	in.
ASCE	Light	30	3 1/8	1 25/64	3 1/8	17/32	11/64	1 11/16	12	21/64	1 23/32	12	3.00	4.10	2.55	—	—
		40	3 1/2	1 71/128	3 1/2	5/8	7/32	1 7/8	12	25/64	1 55/64	12	3.94	6.54	3.59	3.89	1.68
		50	3 7/8	1 23/32	3 7/8	11/16	1/4	2 1/8	12	7/16	2 1/16	12	4.90	10.1	5.10	—	1.88
		60	4 1/4	1 115/128	4 1/4	49/64	9/32	2 3/8	12	31/64	2 17/64	12	5.93	14.6	6.64	7.12	2.05
	—	70	4 5/8	2 3/64	4 5/8	13/16	9/32	2 7/16	12	33/64	2 15/32	12	6.81	19.7	8.19	8.87	2.22
		80	5	2 3/16	5	7/8	19/64	2 1/2	12	35/64	2 5/8	12	7.86	26.4	10.1	11.1	2.38
	Std.	85	5 3/16	2 17/64	5 3/16	57/64	19/64	2 9/16	12	9/16	2 3/4	12	8.33	30.1	11.1	12.2	2.47
		100	5 3/4	2 65/128	5 3/4	31/32	5/16	2 3/4	12	9/16	2 5/64	12	9.84	44.0	14.6	16.1	2.73
ASTM A759	Crane	104	5	2 7/16	5	1 1/16	1/2	2 1/2	12	1	2 7/16	3 1/2	10.3	29.8	10.7	13.5	2.21
		135	5 3/4	2 15/32	5 3/16	1 1/16	15/32	3 7/16	14	1 1/4	2 13/16	12	13.3	50.8	17.3	18.1	2.81
		171	6	2 5/8	6	1 1/4	5/8	4.3	Flat	1 1/4	2 3/4	Vert.	16.8	73.4	24.5	24.4	3.01
		175	6	2 21/32	6	1 9/64	1/2	4 1/4	18	1 1/2	3 7/64	Vert.	17.1	70.5	23.4	23.6	2.98

Table 1–22
ASTM A6 Tolerances for W Shapes and HP Shapes



Permissible Cross-Sectional Variations

Nominal Depth, in.	*A* Depth at Web Centerline, in.		*B* Flange Width, in.		*T* + *T′* Flanges Out of Square, Max. in.	*E*[a] Web Off Center, in.	*C*, Max. Depth at any Cross-Section over Theoretical Depth, in.
	Over	Under	Over	Under			
To 12, incl.	1/8	1/8	1/4	3/16	1/4	3/16	1/4
Over 12	1/8	1/8	1/4	3/16	5/16	3/16	1/4

Permissible Variations in Length

Nominal Depth[b], in.	Variations from Specified Length for Lengths Given, in.			
	30 ft and Under		Over 30 ft	
	Over	Under	Over	Under
Beams 24 in. and under	3/8	3/8	3/8 plus 1/16 for each additional 5 ft or fraction thereof	3/8
Beams over 24 in. All columns	1/2	1/2	1/2 plus 1/16 for each additional 5 ft or fraction thereof	1/2

Mill Straightness Tolerances[c]

Sizes	Length	Permissible Variation from Straight, in.	
		Camber	Sweep
Flange width equal to or greater than 6 in.	All	$\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft)}}{10}$	
Flange width less than 6 in.	All	$\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft)}}{10}$	$\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft)}}{5}$
Certain sections with a flange width approx. equal to depth & specified on order as columns[d]	45 ft and under	$\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft)}}{10}$ with $\frac{3}{8}$ in. max.	
	Over 45 ft	$\frac{3}{8}\text{ in.} + \left[\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft} - 45)}{10}\right]$	

Other Permissible Rolling Variations

Area and Weight	± 2.5 percent theoretical or specified amount.
Ends Out of Square	1/64 in., per in. of depth, or of flange width if it is greater than the depth.

[a] Variation of 5/16 in. max. for sections over 426 lb/ft.

[b] For shapes specified in the order for use as bearing piles, the permitted variations are plus 5 in. and minus 0 in.

[c] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1–1. For tolerance on induced camber and sweep, see Code of Standard Practice Section 6.4.4.

[d] Applies only to W8×31and heavier, W10×49 and heavier, W12×65 and heavier, W14×90 and heavier, HP8×36, HP10×57, HP12×74 and heavier, and HP14×102 and heavier. If other sections are specified on the order as columns, the tolerance will be subject to negotiation with the manufacturer.

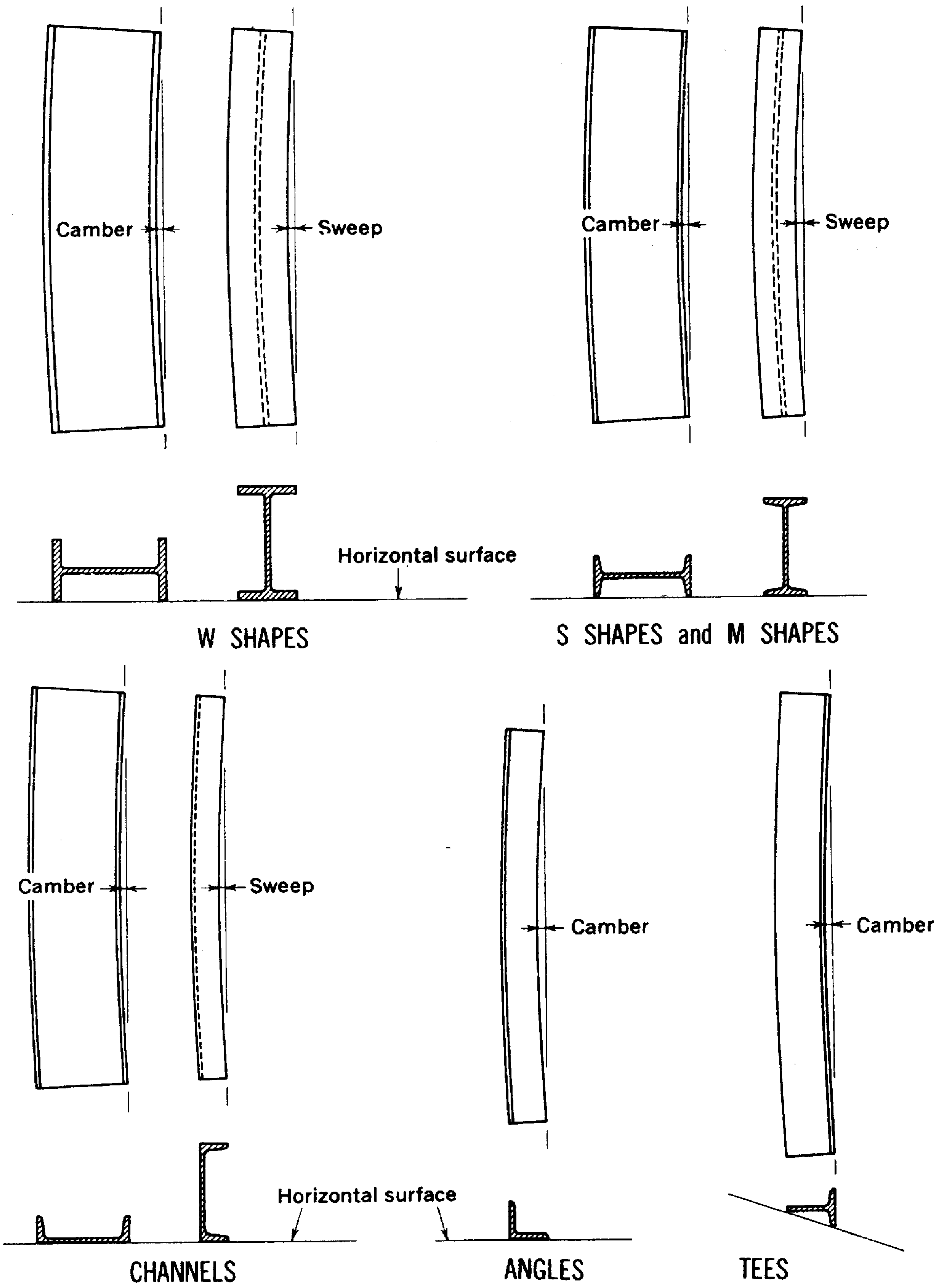
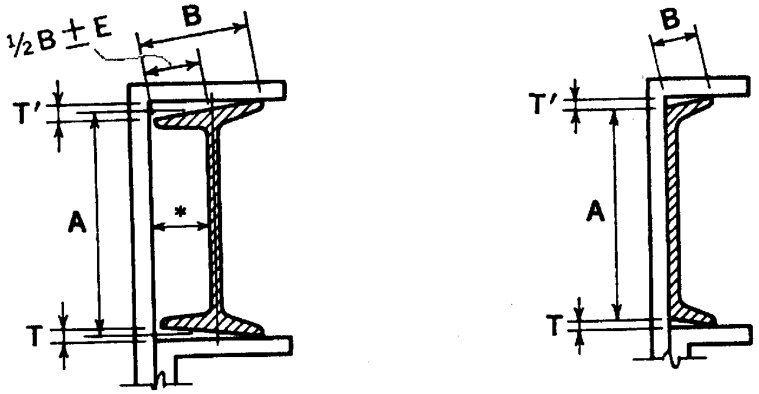


Figure 1–1. Positions for Measuring Straightness.

Table 1–23
ASTM A6 Tolerances for S Shapes, M Shapes, and Channels



*Back of square and centerline of web to be parallel when measuring "out-of-square"

Permissible Cross-Sectional Variations

Shape	Nominal Depth, in.	A[a] Depth, in.		B Flange Width, in.		T + T′[b] Flanges Out of Square, per in. of B, in.	E Web Off Center, in.
		Over	Under	Over	Under		
S shapes and M shapes	3 to 7, incl.	3/32	1/16	1/8	1/8	1/32	3/16
	Over 7 to 14, incl.	1/8	3/32	5/32	5/32		
	Over 14 to 24, incl.	3/16	1/8	3/16	3/16		
Channels	3 to 7, incl.	3/32	1/16	1/8	1/8	1/32	—
	Over 7 to 14, incl.	1/8	3/32	1/8	5/32		
	Over 14	3/16	1/8	1/8	3/16		

Permissible Variations in Length

Shape	Variations from Specified Length for Lengths Given[c], in.					
	5 to 10 ft, excl.	10 to 20 ft, excl.	20 to 30 ft, incl.	Over 30 to 40 ft, incl.	Over 40 to 65 ft, incl.	Over 65 ft
All	1	1 1/2	1 3/4	2 1/4	2 3/4	—

Mill Straightness Tolerances[d]

Camber	$\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft)}}{10}$
Sweep	Due to the extreme variations in flexibility of these shapes, permitted variations for sweep are subject to negotiation between the manufacturer and purchaser for the individual sections involved.

Other Permissible Rolling Variations

Area and Weight	± 2.5 percent theoretical or specified amount.
Ends Out of Square	S Shapes, M Shapes and Channels 1/64 in., per in. of depth.

— Indicates that there is no requirement.

[a] A is measured at center line of web for beams and at back of web for channels.

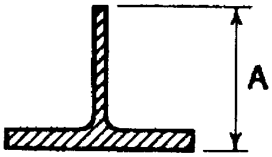
[b] T + T′ applies when flanges of channels are toed in or out.

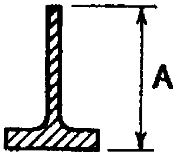
[c] The permitted variation under the specified length is 0 in. for all lengths. There are no requirements for lengths over 65 ft.

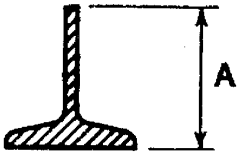
[d] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1–1. For tolerance on induced camber and sweep, see Code of Standard Practice Section 6.4.4.

Table 1–24

ASTM A6 Tolerances for WT, MT, and ST Shapes







Permissible Variations in Depth

Dimension A may be approximately one-half beam depth or any dimension resulting from off-center splitting or splitting on two lines, as specified in the order.

Depth of Shape from which Tee is Split, in.	Variations in Depth A, Over and Under
To 6, excl.	$^1/_8$
6 to 16, excl.	$^3/_{16}$
16 to 20, excl.	$^1/_4$
20 to 24, excl.	$^5/_{16}$
24 and over	$^3/_8$

The above variations in depths of tees include the permissible variations in depth for the beams before splitting

Mill Straightness Tolerances[a]

Camber and Sweep	$^1/_8$ in. × $\frac{\text{(total length, ft)}}{5}$

Other Permissible Rolling Variations

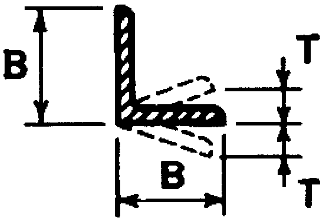
Other permissible variations in cross section as well as permissible variations in length, area, weight, ends out-of-square, and sweep will correspond to those of the beam before splitting.

— Indicates that there is no requirement.

[a] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1–1. For tolerance on induced camber and sweep, see Code of Standard Practice Section 6.4.4.

Table 1–25

ASTM A6 Tolerances for Angles, Structural Size



Permissible Cross-Sectional Variations

Shape	Nominal Leg Size[a], in.	*B* Leg Size, in.		*T* Out of Square per in. of *B*, in.
		Over	Under	
Angles	3 to 4, incl.	1/8	3/32	3/128[b]
	Over 4 to 6, incl.	1/8	1/8	
	Over 6	3/16	1/8	

Permissible Variations in Length

Variations Over Specified Length for Lengths Given[c], in.

5 to 10 ft, excl.	10 to 20 ft, excl.	20 to 30 ft, incl.	Over 30 to 40 ft, incl.	Over 40 to 65 ft, incl.
1	1 1/2	1 3/4	2 1/4	2 3/4

Mill Straightness Tolerances[d]

Camber	$\frac{1}{8}$ in. × $\frac{\text{(total length, ft)}}{5}$, applied to either leg
Sweep	Due to the extreme variations in flexibility of these shapes, permitted variations for sweep are subject to negotiation between the manufacturer and purchaser for the individual sections involved.

Other Permissible Rolling Variations

Area and Weight	± 2.5 percent theoretical or specified amount.
Ends Out of Square	3/128 in. per in. of leg length, or 1 1/2 degrees. Variations based on the longer leg of unequal angle.

[a] For unequal leg angles, longer leg determines classification.

[b] 3/128 in. per in. = 1 1/2 degrees.

[c] The permitted variation under the specified length is 0 in. for all lengths. There are no requirements for lengths over 65 ft.

[d] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1–1. For tolerance on induced camber and sweep, see Code of Standard Practice Section 6.4.4.

Table 1–26
ASTM A6 Tolerances for Angles, Bar Size[a]

Permissible Cross-Sectional Variations

Specified Leg Size[b], in.	Variations in Thickness for Thicknesses Given, Over and Under, in.			*B* Leg Size, Over and Under, in.	*T* Out of Square per Inch of *B*, in.
	3/16 and Under	Over 3/16 to 3/8 incl.	Over 3/8		
1 and Under	0.008	0.010	—	1/32	3/128[c]
Over 1 to 2, incl.	0.010	0.010	0.012	3/64	
Over 2 to 3, excl.	0.012	0.015	0.015	1/16	

Permissible Variations in Length

Section	Variations Over Specified Length for Lengths Given[d], in.				
	5 to 10 ft, excl.	10 to 20 ft, excl.	20 to 30 ft, incl.	Over 30 to 40 ft, incl.	40 to 65 ft, incl.
All bar-size angles	5/8	1	1 1/2	2	2 1/2

Mill Straightness Tolerances[e]

Camber	1/4 in. in any 5 ft, or 1/4 in. × $\frac{(\text{total length, ft})}{5}$, applied to either leg
Sweep	Due to the extreme variations in flexibility of these shapes, permitted variations for sweep are subject to negotiation between the manufacturer and purchaser for the individual sections involved.

Other Permissible Rolling Variations

Ends Out of Square	3/128 in. per in. of leg length, or 1 1/2 degrees. Variations based on the longer leg of unequal angle.

— Indicates that there is no requirement.

[a] A member is "bar size" when its greatest cross-sectional dimension is less than 3 inches.

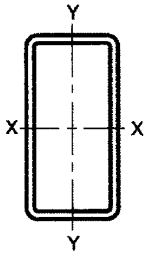
[b] For unequal angles, longer leg determines classification.

[c] 3/128 in. per in. = 1 1/2 degrees.

[d] The permitted variation under the specified length is 0 in. for all lengths. There are no requirements for lengths over 65 ft.

[e] The tolerances herein are taken from ASTM A6 and apply to the straightness of members received from the rolling mill, measured as illustrated in Figure 1–1. For tolerance on induced camber and sweep, see Code of Standard Practice Section 6.4.4.

Table 1–27
Tolerances for Rectangular and Square HSS



<table>
<tr><th colspan="5">ASTM A500, ASTM A501, ASTM A618, and ASTM A847</th></tr>
<tr><td rowspan="6">Outside Dimensions</td><td colspan="4">The outside dimensions, measured across the flats at positions at least 2 in. from either end, shall not vary from the specified dimensions by more than the applicable amount given in the following table:</td></tr>
<tr><td colspan="2">Largest Outside Dimension Across Flats, in.</td><td colspan="2">Permissible Variation Over and Under Specified Dimensions[a,b], in.</td></tr>
<tr><td colspan="2">2½ and under</td><td colspan="2">0.020</td></tr>
<tr><td colspan="2">Over 2½ to 3½, incl.</td><td colspan="2">0.025</td></tr>
<tr><td colspan="2">Over 3½ to 5½, incl.</td><td colspan="2">0.030</td></tr>
<tr><td colspan="2">Over 5½</td><td colspan="2">1 percent[c]</td></tr>
<tr><td rowspan="5">Length</td><td colspan="4">HSS are commonly produced in random lengths, in multiple lengths, and in definite cut lengths. When cut lengths are specified for HSS, the length tolerances shall be in accordance with the following table:</td></tr>
<tr><td colspan="4">Length tolerance for specified cut lengths, in.</td></tr>
<tr><td colspan="2">22 ft and under</td><td colspan="2">Over 22 to 44 ft, incl.</td></tr>
<tr><td>Over</td><td>Under</td><td>Over</td><td>Under</td></tr>
<tr><td>½</td><td>¼</td><td>¾</td><td>¼</td></tr>
<tr><td>Wall Thickness</td><td colspan="4">ASTM A500 and ASTM A847 only: The tolerance for wall thickness exclusive of the weld area shall be plus and minus 10 percent of the nominal wall thickness specified. The wall thickness is to be measured at the center of the flat.</td></tr>
<tr><td>Weight</td><td colspan="4">ASTM A501 only: The weight of HSS, as specified in ASTM A501 Tables 4, 5, and 6, shall not be less than the specified value by more than 3.5 percent.</td></tr>
<tr><td>Mass</td><td colspan="4">ASTM A618 only: The mass shall not be less than the specified value by more than 3.5 percent.</td></tr>
<tr><td>Straightness</td><td colspan="4">The permissible variation for straightness shall be ⅛ in. times the number of ft of total length divided by 5.</td></tr>
<tr><td>Squareness of Sides</td><td colspan="4">Adjacent sides may deviate from 90 degrees by a tolerance of plus or minus 2 degrees maximum.</td></tr>
<tr><td>Radius of Corners</td><td colspan="4">The radius of any outside corner of the section shall not exceed 3 times the specified wall thickness[d].</td></tr>
<tr><td rowspan="9">Twist</td><td colspan="4">The tolerances for twist with respect to axial alignment of the section shall be as shown in the following table:</td></tr>
<tr><td colspan="2">Specified Dimension of Longest Side, in.</td><td colspan="2">Maximum Twist per 3 ft and in Each Additional 3 ft, in.</td></tr>
<tr><td colspan="2">1½ and under</td><td colspan="2">0.050</td></tr>
<tr><td colspan="2">Over 1½ to 2½, incl.</td><td colspan="2">0.062</td></tr>
<tr><td colspan="2">Over 2½ to 4, incl.</td><td colspan="2">0.075</td></tr>
<tr><td colspan="2">Over 4 to 6, incl.</td><td colspan="2">0.087</td></tr>
<tr><td colspan="2">Over 6 to 8, incl.</td><td colspan="2">0.100</td></tr>
<tr><td colspan="2">Over 8</td><td colspan="2">0.112</td></tr>
<tr><td colspan="4">Twist shall be determined by holding one end of the HSS down on a flat surface plate, measuring the height that each corner on the bottom side of the tubing extends above the surface plate near the opposite ends of the HSS, and calculating the difference in the measured heights of such corners.</td></tr>
</table>

[a] The respective outside dimension tolerances include the allowances for convexity and concavity.

[b] ASTM A500 and ASTM A847 HSS only: The tolerances given are for the large flat dimension only. For HSS having a ratio of outside large to small flat dimension less than 1.5, the tolerance on the small flat dimesion shall be identical to those given. For HSS having a ratio of outside large to small flat dimension in the range of 1.5 to 3.0 inclusive, the tolerance on the small flat dimesion shall be 1.5 times those given. For HSS having a ratio of outside large to small flat dimension greater than 3.0, the tolerance on the small flat dimension shall be 2.0 times those given.

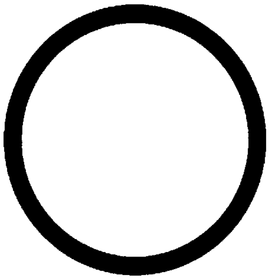
[c] ASTM A500 HSS only: This value is 0.1 times the large flat dimension.

[d] ASTM A501 HSS only: The radius of any outside corner must not exceed 3 times the calculated nominal wall thickness.

[e] ASTM A500, ASTM A501, and ASTM A847 HSS only: For heavier sections it shall be permissible to use a suitable measuring device to determine twist. Twist measurements shall not be taken within 2 in. of the ends of the HSS.

Table 1–28

Tolerances for Round HSS and Pipe



<table>
<tr><th colspan="5">ASTM A53</th></tr>
<tr><td>Weight</td><td colspan="4">The weight as specified in ASTM A53 Table X2.2 and Table X2.3 or as calculated from the relevant equation in ANSI/ASME B36.10M shall not vary by more than ±10 percent. Note that the weight tolerance is determined from the weights of the customary lifts of pipe as produced for shipment by the mill, divided by the number of ft of pipe in the lift. On pipe sizes over 4 in. where individual lengths may be weighed, the weight tolerance is applicable to the individual length.</td></tr>
<tr><td>Diameter</td><td colspan="4">For pipe 2 in. and over in nominal diameter, the outside diameter shall not vary more than ± 1 percent from the standard specified.</td></tr>
<tr><td>Thickness</td><td colspan="4">The minimum wall thickness at any point shall not be more than 12.5 percent under the nominal wall thickness specified.</td></tr>
<tr><th colspan="5">ASTM A500 and ASTM A847</th></tr>
<tr><td>Diameter[a]</td><td colspan="4">For HSS 1.900 in. and under in nominal diameter, the outside diameter shall not vary more than ± 0.5 percent, rounded to the nearest 0.005 in., from the nominal diameter specified.
For HSS 2.000 in. and over in nominal diameter, the outside diameter shall not vary more than ± 0.75 percent, rounded to the nearest 0.005 in., from the nominal diameter specified.</td></tr>
<tr><td>Thickness</td><td colspan="4">The wall thickness at any point, excluding the weld seam of welded tubing, shall not be more than 10 percent under or over the nominal wall thickness specified.</td></tr>
<tr><th colspan="5">ASTM A501 and ASTM A618</th></tr>
<tr><td>Outside Dimensions</td><td colspan="4">For HSS 1 1/2 inches and under in nominal size, the outside diameter shall not vary more than 1/64 in. over nor more than 1/32 in. under the specified diameter.
For round hot-formed HSS 2 in. and over in nominal size, the outside diameter shall not vary more than ± 1 percent from the specified diameter.</td></tr>
<tr><td>Weight (A501 only)</td><td colspan="4">The weight of HSS, as specified in ASTM A501 Tables 4, 5, and 6, shall not be less than the specifie value by more than 3.5 percent.</td></tr>
<tr><td>Mass (A618 only)</td><td colspan="4">The mass of HSS shall not be less than the specified value by more than 3.5 percent. The mass tolerance shall be determined from individual lengths or, for HSS 4 1/2 in. and under in nominal size, shall be determined from masses of customary lifts produced by the mill.</td></tr>
<tr><th colspan="5">ASTM A500, ASTM A501, ASTM A618 and ASTM A847</th></tr>
<tr><td rowspan="5">Length</td><td colspan="4">HSS are commonly produced in random mill lengths, in multiple lengths, and in definite cut lengths. When cut lengths are specified for HSS, the length tolerances shall be in accordance with the following table:</td></tr>
<tr><td colspan="4">Length tolerance for specified cut lengths, in.</td></tr>
<tr><td colspan="2">22 ft and under</td><td colspan="2">Over 22 to 44 ft, incl.</td></tr>
<tr><td>Over</td><td>Under</td><td>Over</td><td>Under</td></tr>
<tr><td>1/2</td><td>1/4</td><td>3/4</td><td>1/4</td></tr>
<tr><td>Straightness</td><td colspan="4">The permissible variation for straightness of HSS shall be 1/8 in. times the number of ft of total length divided by 5.</td></tr>
<tr><td colspan="5">[a] The outside diameter measurements shall be taken at least 2 in. from the end of the HSS.</td></tr>
</table>

Table 1–29

Rectangular Sheared Plates

Permissible Variations from Flatness

(Carbon Steel Only)

Specified Thickness, in.	Variations from Flatness for Specified Widths, in.							
	To 36, excl.	36 to 48, excl.	48 to 60, excl.	60 to 72, excl.	72 to 84, excl.	84 to 96, excl.	96 to 108, excl.	108 to 120, excl.
To 1/4, excl.	9/16	3/4	15/16	1 1/4	1 3/8	1 1/2	1 5/8	1 3/4
1/4 to 3/8, excl.	1/2	5/8	3/4	15/16	1 1/8	1 1/4	1 3/8	1 1/2
3/8 to 1/2, excl.	1/2	9/16	5/8	5/8	3/4	7/8	1	1 1/8
1/2 to 3/4, excl.	7/16	1/2	9/16	5/8	5/8	3/4	1	1
3/4 to 1, excl.	7/16	1/2	9/16	5/8	5/8	5/8	3/4	7/8
1 to 2, excl.	3/8	1/2	1/2	9/16	9/16	5/8	5/8	5/8
2 to 4, excl.	5/16	3/8	7/16	1/2	1/2	1/2	1/2	9/16
4 to 6, excl.	3/8	7/16	1/2	1/2	9/16	9/16	5/8	3/4
6 to 8, excl.	7/16	1/2	1/2	5/8	11/16	3/4	7/8	7/8

Notes:

1. The longer dimension specified is considered the length, and permissible variations in flatness along the length should not exceed the tabular amount for the specified width in plates up to 12 ft in length, or in any 12 ft for longer plates.
2. The flatness variations across the width should not exceed the tabular amount for the specified width.
3. When the longer dimension is under 36 in., the permissible variation should not exceed 1/4 in. When the longer dimension is from 36 to 72 in., inclusive, the permissible variation should not exceed 75 percent of the tabular amount for the specified width, but in no case less than 1/4 in.
4. These variations apply to plates which have a specified minimum tensile strength of not more than 60 ksi or comparable chemistry or hardness. The limits in the table are increased 50 percent for plates specified to a higher minimum tensile strength or comparable chemistry or hardness.
5. For plates 8 in. and over in thickness or 120 in. and over in width, see ASTM A6 Table 13.

Permissible Variations in Camber[a] for Carbon Steel Sheared and Gas Cut Rectangular Plates

Maximum permissible camber, in. (all thicknesses) = $\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft)}}{5}$

Permissible Variations in in Camber[a] for High-Strength Low-Alloy and Alloy Steel Sheared, Special-Cut, or Gas-Cut Rectangular Plates

Dimension, in.		Camber for Thicknesses and Widths Given
Thickness	Width	
To 2, incl.	All	$\frac{1}{8}\text{ in.} \times \frac{\text{(total length, ft)}}{5}$
Over 2 to 15, incl.	To 30, incl.	$\frac{3}{16}\text{ in.} \times \frac{\text{(total length, ft)}}{5}$
	Over 30 to 60, incl.	$\frac{1}{4}\text{ in.} \times \frac{\text{(total length, ft)}}{5}$

[a] Camber as it relates to plates is the horizontal edge curvature in the length, measured over the entire length of the plate in the flat position.

PART 2

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SCOPE

The specification requirements and other design considerations summarized in this Part apply in general to the design and construction of steel buildings. For seismic force resisting systems in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

APPLICABLE SPECIFICATIONS, CODES, AND STANDARDS

Specifications, Codes, and Standards for Structural Steel Buildings

Subject to the requirements in the applicable building code and the contract documents, the design, fabrication, and erection of structural steel buildings is governed as indicated in the AISC Specification Sections A1 and B2 as follows:

1. ASCE 7: *Minimum Design Loads for Buildings and Other Structures,* SEI/ASCE 7-02. Available from the American Society of Civil Engineers, ASCE 7 provides the general requirements for loads, load factors, and load combinations.
2. AISC Specification: The 2005 AISC *Specification for Structural Steel Buildings*, included in Part 16 of this Manual and available at www.aisc.org, provides the general requirements for design and construction.
3. Code of Standard Practice: The 2005 AISC *Code of Standard Practice for Steel Buildings and Bridges*, included in Part 16 of this manual and available at www.aisc.org, provides the standard of custom and usage for the fabrication and erection of structural steel.

Other referenced standards include:

1. RCSC Specification: The 2004 RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, reprinted in Part 16 of this Manual with the permission of the Research Council on Structural Connections and available at www.boltcouncil.org, provides the additional requirements specific to bolted joints with high-strength bolts.
2. AWS D1.1: *Structural Welding Code—Steel,* AWS D1.1:2004. Available from the American Welding Society, AWS D1.1 provides additional requirements specific to welded joints. Requirements for the proper specification of welds can be found in AWS A2.4: *Standard Symbols for Welding, Brazing, and Nondestructive Examination.*
3. ACI 318: *Building Code Requirements for Structural Concrete.* Available from the American Concrete Institute, ACI 318 provides additional requirements for reinforced concrete, including in composite design and the design of steel-to-concrete anchorage.

Various other specifications and standards from ASME, ASTM, and ACI are also referenced in AISC Specification Section A2.

Additional Requirements for Seismic Applications

The 2005 AISC *Seismic Provisions for Structural Steel Buildings* apply when the seismic response modification factor, R, is taken greater than 3, or when required by the applicable building code. This specification is available in Part 6 of the AISC *Seismic Design Manual* and at www.aisc.org. When R is taken equal to or less than 3, these additional requirements do not apply.

Other AISC Reference Documents

The following other AISC publications may be of use in the design and construction of structural steel buildings:

1. AISC *Design Examples* is a CD-based companion to this Manual and includes design examples outlining the application of design aids and Specification provisions developed in coordination with this Manual.
2. AISC's *Detailing for Steel Construction*, Second Edition, covers the standard practices and recommendations for steel detailing, including preparation of shop and erection drawings.
3. The AISC *Seismic Design Manual* provides guidance on steel design in seismic applications, in accordance with the AISC Seismic Provisions.

Additionally, the following AISC Design Guides are available at www.aisc.org for in-depth coverage of specific topics in steel design:

1. *Column Base Plates*, AISC Design Guide No. 1 (DeWolf, 1990).
2. *Design of Steel and Composite Buildings with Web Openings*, AISC Design Guide No. 2 (Darwin, 1990).
3. *Serviceability Design Considerations for Low-Rise Buildings*, AISC Design Guide No. 3 (West and Fisher, 2003).
4. *Extended End-Plate Moment Connections*, AISC Design Guide No. 4 (Murray, 2004).
5. *Design of Low- and Medium-Rise Steel Buildings*, AISC Design Guide No. 5 (Allison, 1991).
6. *Load and Resistance Factor Design of W-Shapes Encased in Concrete*, AISC Design Guide No. 6 (Griffis, 1992).
7. *Industrial Buildings: Roofs to Column Anchorage*, AISC Design Guide No. 7 (Fisher, 2005).
8. *Partially Restrained Composite Connections*, AISC Design Guide No. 8 (Leon, Hoffman, and Staeger, 1996).
9. *Torsional Analysis of Structural Steel Members*, AISC Design Guide No. 9 (Seaburg and Carter, 1997).
10. *Erection Bracing of Low-Rise Structural Steel Frames*, AISC Design Guide No. 10 (Fisher and West, 1997).
11. *Floor Vibrations Due to Human Activity*, AISC Design Guide No. 11 (Murray, Allen and Ungar, 1997).
12. *Modification of Existing Welded Steel Moment Frames for Seismic Resistance*, AISC Design Guide No. 12 (Gross, Engelhardt, Uang, Kasai, and Iwankiw, 1999).
13. *Wide-Flange Column Stiffening at Moment Connections: Wind and Seismic Applications*, AISC Design Guide No. 13 (Carter, 1999).
14. *Staggered Truss Framing Systems,* AISC Design Guide No. 14 (Wexler and Lin, 2001).
15. *AISC Rehabilitation and Retrofit Guide: A Reference for Historic Shapes and Specifications*, AISC Design Guide No. 15 (Brockenbrough, 2002).
16. *Flush and Extended Multiple-Row Moment End-Plate Connections*, AISC Design Guide No. 16 (Murray and Shoemaker, 2002).

17. *High-Strength Bolts—A Primer for Structural Engineers,* AISC Design Guide No. 17 (Kulak, 2002).
18. *Steel-Framed Open-Deck Parking Structures,* AISC Design Guide No. 18 (Churches, Troup, and Angeloff, 2003).
19. *Fire Resistance of Structural Steel Framing,* AISC Design Guide No. 19 (Ruddy, Marlo, Ioannides, and Alfawakhiri, 2003).

OSHA REQUIREMENTS

OSHA Safety and Health Standards for the Construction Industry, 29 CFR 1926 Part R Safety Standards for Steel Erection, must be addressed in the design, detailing, fabrication, and erection of steel structures. These regulations became effective on July 18, 2001, except for requirements for slip-resistance certification of painted surfaces (see "Walking/Working Surfaces" below), which are expected to become effective on July 18, 2006. A brief summary of selected provisions is available (Barger and West, 2001). The full text of the regulations should be consulted and can be found at www.osha.gov.

USING THE 2005 AISC SPECIFICATION

The 2005 AISC *Specification for Structural Steel Buildings* (AISC 360-05) unifies the design provisions formerly presented in the 1989 *Specification for Structural Steel Buildings: Allowable Stress Design and Plastic Design* and the 1999 *Load and Resistance Factor Design Specification for Structural Steel Buildings.* It also integrates into a single document the information previously provided in the 1993 *Load and Resistance Factor Design Specification for Single-Angle Members* and the 1997 *Specification for the Design of Steel Hollow Structural Sections.* This new unified specification, in combination with the 2005 *Seismic Provisions for Structural Steel Buildings* (AISC 341-05), brings together all of the provisions needed for the design of structural steel in buildings and other structures.

The 2005 AISC Specification presents two approaches for the design of structural steel members and connections. Chapter B establishes the general requirements for analysis and design. It states that, "designs shall be made according to the provisions for Load and Resistance Factor Design (LRFD) or to the provisions for Allowable Strength Design (ASD)." These two approaches are equally valid for any structure for which the Specification is applicable. There is no preference stated or implied in the provisions.

The required strength of structural members and connections may be determined by elastic, inelastic, or plastic analysis for the load combinations associated with either LRFD or ASD and as stipulated by the applicable building code. In all cases, the available strength must exceed the required strength. The AISC Specification gives provisions for determining the available strength as summarized below.

Load and Resistance Factor Design (LRFD)

Load and Resistance Factor Design according to the 2005 AISC Specification is essentially the same as LRFD according to the previous three LRFD specifications. Although some of the provisions have changed from previous specifications, the overall approach has remained constant. If there is a desire to use the LRFD provisions in the form of stresses, the strength provisions can be transformed into stress provisions by factoring out the appropriate section property.

The load combinations appropriate for LRFD are given in the applicable building code or, in its absence, ASCE 7 Section 2.3. For LRFD, the available strength is referred to as the design strength. All of the LRFD provisions are structured so that the design strength must equal or exceed the required strength. This is presented in Section B3.3 as

$$R_u \leq \phi R_n$$

In this equation, R_u is the required strength determined by analysis for the LRFD load combinations, R_n is the nominal strength determined according to the specification provisions, and ϕ is the resistance factor given by the specification for a particular limit-state. Throughout this Manual, tabulated values of ϕR_n, the design strength, are given for LRFD. These values are tabulated as blue numbers in columns with the heading LRFD.

Allowable Strength Design (ASD)

Allowable Strength Design is similar to what is known as Allowable Stress Design in that they are both carried out at the same load level. Thus, the same load combinations are used. The difference is that for strength design, the primary provisions are given in terms of forces or moments rather than stresses. In every situation, these strength provisions can be transformed into stress provisions by factoring out the appropriate section property.

The load combinations appropriate for ASD are given by the applicable building code or, in its absence, ASCE 7 Section 2.4. For ASD, the available strength is referred to as the allowable strength. All of the ASD provisions are structured so that the allowable strength must equal or exceed the required strength. This is presented in Section B3.4 as

$$R_a \leq R_n/\Omega$$

In this equation, R_a is the required strength determined by analysis for the ASD load combinations, R_n is the nominal strength determined according to the specification provisions, and Ω is the safety factor given by the specification for a particular limit-state. Throughout this Manual, tabulated values of R_n/Ω, the allowable strength, are given for ASD. These values are tabulated as black numbers on a green background in columns with the heading ASD.

DESIGN FUNDAMENTALS

It is commonly believed that ASD was an elastic design method based entirely in a stress format without limit-states and LRFD was an inelastic design method based entirely in a strength format with limit-states. Traditional ASD was based in limit-states principles too, but without the use of the term. Additionally, either method can be formulated in a stress or strength basis, and both take advantage of inelastic behavior. The 2005 AISC Specification highlights how similar LRFD and ASD are in its formulation, with identical provisions throughout for LRFD and ASD.

Design according to the 2005 Specification, whether it is according to LRFD or ASD, is based on limit states design principles, which define the boundaries of structural usefulness. Strength limit states relate to load carrying capability and safety. Serviceability limit-states relate to performance under normal service conditions. Structures must be proportioned so that no applicable strength or serviceability limit-state is exceeded.

Normally, several limit-states will apply in the determination of the nominal strength of a structural member or connection. The controlling limit-state is normally the one that results in the least available strength. As an example, the controlling limit-state for bending of a simple beam may be yielding, local buckling, or lateral-torsional buckling for strength, or deflection or vibration for serviceability. The tabulated values may either reflect a single limit-state or a combination of several limit-states. This will be clearly stated in the introduction to the particular tables.

Loads, Load Factors, and Load Combinations

Based on Specification Sections B3.3 and B3.4, the required strength (either P_u, M_u, V_u, etc. for LRFD or P_a, M_a, V_a, etc. for ASD) is determined for the appropriate load magnitudes, load factors, and load combinations given in the applicable building code. These are usually based on ASCE-7, which may be used when there is no applicable building code. The common loads found in building structures are:

D = dead load
L = live load due to occupancy
L_r = roof live load
S = snow load
R = nominal load due to initial rainwater or ice exclusive of the ponding contribution
W = wind load
E = earthquake load

Load and Resistance Factor Design

For LRFD, the required strength is determined from the following factored combinations[1], which are based on ASCE-7 Section 2.3:

$$1.4D \quad (1)$$
$$1.2D + 1.6L + 0.5(L_r \text{ or } S \text{ or } R) \quad (2)$$
$$1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (0.5L \text{ or } 0.8W) \quad (3)$$
$$1.2D + 1.6W + 0.5L + 0.5(L_r \text{ or } S \text{ or } R) \quad (4)$$
$$1.2D \pm 1.0E + 0.5L + 0.2S \quad (5)$$
$$0.9D \pm (1.6W \text{ or } 1.0E) \quad (6)$$

The load combinations for LRFD recognize that, when several transient loads act in combination, only one assumes its maximum lifetime value[2], while the other(s) are at their "arbitrary-point-in-time" (APT) values. Each combination models the total design loading condition when a different load is at its maximum. Thus, the maximum-lifetime load effect is amplified by an amount that is proportional to its relative variability and the APT load effect(s) are factored to their mean value(s). With this approach, the margin of safety varies with the load combination yielding a more uniform reliability than would be expected when nominal loads are combined directly.

Dead load, D, is present in each load combination with a load factor of 1.2, except in load combination 1, where it is the dominant (only) load effect, and load combination 6, where it is reduced for calculation of the overturning or uplift effect. The 1.2 load factor accounts

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[1] Exception: Per ASCE 7, the load factor on L in combinations 3, 4, and 5 shall equal 1.0 for garages, areas occupied as places of public assembly, and all areas where the live load is greater than 100 psf.

[2] Usually based upon a 50-year recurrence, except for seismic loads.

for the statistical variability of the dead load. The designer must independently account for other contributions to dead load, such as the weight of additional concrete, if any, added to adjust for concrete ponding effects (Ruddy, 1986) or differing framing elevations.

Allowable Strength Design

For ASD, the required strength is determined from the following combinations, which are also based on ASCE-7 Section 2.4:

$$D \qquad (1)$$
$$D + L \qquad (2)$$
$$D + (L_r \text{ or } S \text{ or } R) \qquad (3)$$
$$D + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) \qquad (4)$$
$$D \pm (W \text{ or } 0.7E) \qquad (5)$$
$$D + 0.75(W \text{ or } 0.7E) + 0.75L + 0.75(L_r \text{ or } S \text{ or } R) \qquad (6)$$
$$0.6D \pm (W \text{ or } 0.7E) \qquad (7)$$

The load combinations for ASD combine the code-specified nominal loads directly with no factors for those cases where loads with minimal variation with time are combined, cases 1, 2, and 3. For those cases where multiple time-variable loads are included, a 0.75 reduction factor is applied to the time-variable loads only. Since all of the safety in an ASD design comes through the introduction of the safety factor on the resistance side of the equation, each load case uses the same safety factor for a given limit-state.

In ASD, when considering members subjected to gravity loading only, it is clear that the controlling load combination is the one that adds the larger live load to the dead load. Thus, for a floor that does not carry roof load, the controlling combination will be $D + L$, while, for a roof, the controlling combination will be $D + (L_r \text{ or } S \text{ or } R)$. For gravity columns, after live load reductions have been accounted for, the floor and roof live loads may be reduced to 0.75 of their nominal values. A similar reduction is permitted for live loads in combination with lateral loads.

Superposition of Loads in Load Combinations

Whether the loads themselves or the effects of those loads are used in these combinations, LRFD or ASD, the results are the same, provided the principle of superposition is valid. This is true when deflections are small and the stress-strain behavior is nominally elastic. However, when second-order effects are significant or the behavior is inelastic, superposition is not valid and the loads, rather than the load effects, should be used in these combinations.

Nominal Strengths, Resistance Factors, Safety Factors, and Available Strengths

The 2005 AISC Specification requires that the available strength must be greater than the required strength for any element. The available strength is a function of the nominal strength given by the specification and the corresponding resistance factor or safety factor. As discussed earlier, the required strength can be determined either with LRFD or ASD load combinations.

The available strength for LRFD is the design strength, which is calculated as the product of the resistance factor ϕ and the nominal strength (ϕP_n, ϕM_n, ϕV_n, etc.) The available strength for ASD is the allowable strength, which is calculated as the quotient of the nominal strength and the corresponding safety factor Ω (P_n/Ω, M_n/Ω, V_n/Ω, etc.)

In LRFD, the margin of safety for the loads is contained in the load factors, and resistance factors, ϕ, account for unavoidable variations in materials, design equations, fabrication, and erection. In ASD, a single margin of safety for all of these effects is contained in the safety factor, Ω.

The resistance factors, ϕ, and safety factors, Ω, in the AISC Specification are based upon research (Galambos et al., 1978), and the experience and judgment of the AISC Committee on Specifications. In general, ϕ is less than unity and Ω is greater than unity. The higher the variability in the test data for a given nominal strength, the lower its ϕ factor and the higher its Ω factor will be. Some examples of ϕ and Ω factors for steel members are as follows:

ϕ = 0.90 for limit-states involving yielding
ϕ = 0.75 for limit-states involving rupture
Ω = 1.67 for limit-states involving yielding
Ω = 2.00 for limit-states involving rupture

The general relationship between the safety factor, Ω, and the resistance factor, ϕ, is

$$\Omega = \frac{1.5}{\phi}$$

Serviceability

Serviceability requirements of the 2005 AISC Specification are found in Section B3.7 and Chapter L. The serviceability limit-states should be selected appropriately for the specific application, as discussed in the Specification Commentary to Chapter L. Serviceability limit-states and the appropriate load combinations for checking their conformance to serviceability requirements can be found in ASCE 7 Appendix B and its Commentary. It should be noted that the load combinations in ASCE 7 Sections 2.3 for LRFD and 2.4 for ASD are both for strength design, and are not necessarily appropriate for consideration of serviceability.

Guidance is also available in the Commentary on the 2005 AISC Specification, both in general and for specific criteria, including camber, deflection, drift, vibrations, wind induced motion, expansion and contraction, and connection slip. Additionally, the applicable building code may provide some further guidance or establish requirements. See also the serviceability discussions in Parts 3 through 6, AISC Design Guide No. 3 *Serviceability Design Considerations for Steel Buildings* (Fisher and West, 2004), and AISC Design Guide No. 11 *Floor Vibrations Due to Human Activity* (Murray et al., 1997).

Required Strength, Stability, Effective Length, and Second-Order Effects

As previously discussed, the Specification requires that the required strength must be less than or equal to the available strength in the design of every member and connection. Chapter C also requires that stability shall be provided for the structure as a whole and each of its elements. Any method that considers the influence of second-order effects, also known as P-delta effects, may be used. Thus, required strengths must be determined including second-order effects, as described in Specification Section C2.1 or Appendix 7. Note that Specification Section C2.1 and Appendix 7 permit an amplified first-order analysis as one method of second-order analysis.

Second-order effects are the additional forces, moments, and displacements resulting from the applied loads acting in their displaced positions as well as the changes from the undeformed to the deformed geometry of the structure. Second-order effects are obtained by considering equilibrium of the structure within its deformed geometry. There are numerous ways of accounting for these effects. The commentary to AISC Specification Appendix 7 provides some guidance on methods of second-order analysis and suggests several benchmark problems for checking the adequacy of analysis methods.

Since the mid-1960s, there have been provisions in the AISC Specifications to account for second-order effects. Initially, these provisions were embedded in the interaction equations. In past ASD Specifications, second-order effects were accounted for by the term

$$\frac{1}{1-\frac{f_a}{F'_e}}$$

found in the interaction equation. In the previous three LRFD Specifications, the factors B_1 and B_2 from Chapter C of those specifications were used to amplify moments to account for second-order effects. B_1 was used to account for the second-order effects due to member curvature and B_2 was used to account for second-order effects due to sidesway. In both specifications, more exact methods were permitted.

The 2005 AISC Specification fully integrates the provisions for stability design with specified methods of second-order analysis. Section C1.3a provides that in braced frames, the effective length factor, K, may be taken as 1.0 and Section C1.3c provides that for gravity-only framing systems, K may also be taken as 1.0. For moment frames, Section C1.3b requires that a critical buckling analysis be performed according to Section C2. The determination of effective length is directly linked to the approach taken for second-order analysis. This is discussed in more detail in Commentary Section C2. Section C2 of the Specification details the requirements for determination of required strengths and, along with Appendix 7, provides three approaches that may be followed.

- The *Direct Analysis Method* is provided in Appendix 7. This is the most comprehensive and, as the name suggests, most direct approach to incorporating all necessary factors in the analysis. Through the use of notional loads, reduced stiffness, and a second-order analysis, the design can be carried out with the forces and moments from the analysis and an effective length equal to the member length, $K = 1.0$.
- The *Effective Length Method* is given in Section C2.2a. In this method, all gravity-only load cases have a minimum lateral load equal to 0.2 percent of the story gravity load applied. A second-order analysis is carried out and, depending on the ratio of the second-order drift to the first-order drift, the effective length may be taken as the member length, $K = 1.0$, or may have to be determined from analysis.
- The *First-Order Analysis Method* is given in Section C2.2b. With this approach, second-order effects are captured through the application of an additional lateral load equal to at least 0.42 percent of the story gravity load applied in each load case. No further second-order analysis is necessary. The required strengths are taken as the forces and moments obtained from the analysis and the effective length factor is $K = 1.0$.

When a second-order analysis is called for in the above methods, Section C2.1a allows any method that properly considers P-delta effects. This may be a true second-order analysis or a simplified approach. One such method is the Amplified First-Order Elastic Analysis provided in Section C2.1b. This is a modified carry over of the B_1/B_2 approach used in the previous LRFD Specification, which was an extension of the simple approach taken in past ASD Specifications.

Simplified Determination of Required Strength

The features of each of the foregoing methods are summarized and compared in Table 2–1. When a fast, conservative solution is desired, the following simplification of the Effective Length Method can be used.

The Effective Length Method and the Amplified First-Order Elastic Analysis approach of Section C2.1b can be used to accomplish the second-order analysis. The User Note in Section C2.1b indicates that for members where the member amplification (P-δ) factor is small, that is, $B_1 \leq 1.05$, it is conservative to amplify the total moment and force by B_2. Thus, equations C2-1a and C2-1b become

LRFD	ASD
$M_r = B_1 M_{nt} + B_2 M_{lt} = B_2 M_u$ $P_r = P_{nt} + B_2 P_{lt} = B_2 P_u$	$M_r = B_1 M_{nt} + B_2 M_{lt} = B_2 M_a$ $P_r = P_{nt} + B_2 P_{lt} = B_2 P_a$

To use this Simplified Method, B_1 should not exceed B_2. For members not subject to transverse loading between their ends, it is very unlikely that B_1 would be greater than 1.0. In addition, the simplified approach is not valid if the amplification factor $B_2 > 1.5$. It is up to the engineer to ensure that the frame is proportioned appropriately to use this simplified approach. In most designs, it is not advisable to have a final structure where the second-order amplification is greater than 1.5, although it is acceptable. In those cases, one should consider stiffening the structure.

Step 1: Perform a first-order elastic analysis. Gravity load cases must include a minimum lateral load at each story equal to 0.002 times the story gravity load where the story gravity load is the load introduced at that story, independent of any loads from above.

Step 2: Establish the design story drift limit and determine the lateral load that produces that drift. This is intended to be a measure of the lateral stiffness of the structure.

Step 3: Determine the ratio of the total story gravity load to the lateral load determined in Step 2. For an ASD design, this ratio must be multiplied by 1.6 before entering the Table below.

Design Story Drift Limit	**Load Ratio from Step 3 (times 1.6 for ASD, 1.0 for LRFD)**										
	0	**5**	**10**	**20**	**30**	**40**	**50**	**60**	**80**	**100**	**120**
H/100	1	1.1	1.1	1.3	1.5	–	–	–	–	–	–
H/200	1	1	1.1	1.1	1.2	1.3	1.4	1.5	–	–	–
H/300	1	1	1	1.1	1.1	1.2	1.2	1.3	1.5	–	–
H/400	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4	1.5
H/500	1	1	1	1	1.1	1.1	1.1	1.2	1.2	1.3	1.4

Step 4: Multiply all of the forces and moments from the first-order analysis by the value obtained from the Table below. Use the resulting forces and moments as the required strengths for the designs of all members and connections.

Step 5: For all cases where the multiplier is 1.1 or less, shown shaded in the table above, the effective length may be taken as the member length, $K = 1.0$. For cases where the multiplier is greater than 1.1 but does not exceed 1.5, determine the effective length factor through analysis, such as with the alignment charts of the Commentary. For cases where no value is shown for the multiplier, the structure must be stiffened in order to use this simplified approach.

Step 6: Ensure that the drift limit set in Step 2 is not exceeded and revise design as needed.

STABILITY BRACING

Beams, girders, and trusses must be restrained against rotation about their longitudinal axes at points of support (a basic assumption stated in the preamble of Specification Chapter F). Additionally, stability bracing with adequate strength and stiffness must be provided consistent with that assumed at braced points in the analysis for frames, columns, and beams (see Appendix 6). Some guidance for special cases follows:

Simple-Span Beams

In general, adequate lateral bracing is provided to the compression flange of a simple-span beam by the connections of infill beams, joists, concrete slabs, metal deck, concrete slabs on metal deck, and similar framing elements.

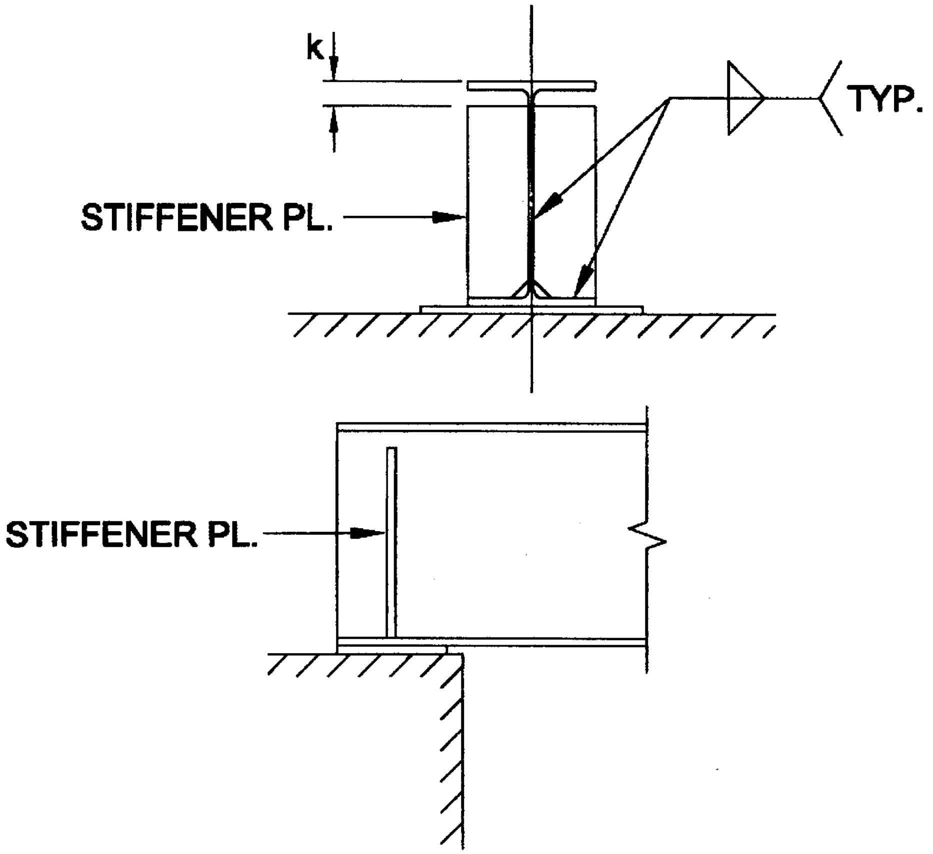
Beam Ends Supported on Bearing Plates

The stability of a beam end supported on a bearing plate can be provided in one of several ways (see Figure 2-1):

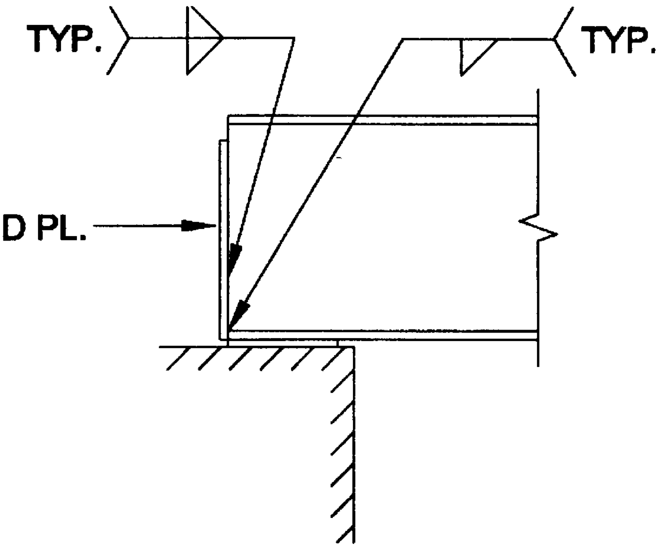
1. The beam end can be built into solid concrete or masonry using anchorage devices.
2. The beam top flange can be stabilized through interconnection with a floor or roof system, provided that system is itself anchored to prevent its translation relative to the beam bearing.
3. A top-flange stability connection can be provided.
4. An end-plate or transverse stiffeners located over the bearing plate extending to near the top-flange *k*-distance can be provided. Such stiffeners must be welded to the top of the bottom flange and to the beam web, but need not extend to or be welded to the top flange.

In each case, the beam and bearing plate must also be anchored to the support. For the design of beam bearing plates, see Part 14.

In atypical framing situations, such as when very deep beams are used, the strength and stiffness requirements in AISC Specification Appendix 6 can be applied to ensure the stability of the assembly. It may also be possible to demonstrate in a limited number of cases, such as with beams with thick webs and relatively shallow depths, that the beam has been properly designed without providing the details described above. In this case, the beam and bearing plate must still be anchored to the support. In any case, it should be noted that the assembly must also meet the requirements in AISC Specification Section J11.



(a) Stability provided with transverse stiffeners.



(b) Stability provided with an end-plate.

ANCHOR BEAM AND/OR
BEARING PL. AS REQUIRED

Figure 2–1. Beam end supported on bearing plate.

Beams and Girders Framing Continuously Over Columns

Roof framing is commonly configured with cantilevered beams that frame continuously over the tops of columns to support drop-in beams between the cantilevered segments (Rongoe, 1996; CISC, 1989). It is also commonly desirable to provide an assembly in which the intersection of the beam and column can be considered a braced point for the design of both the continuous cantilevering beam and the column top. The required stability can be provided in several ways (see Figure 2–2):

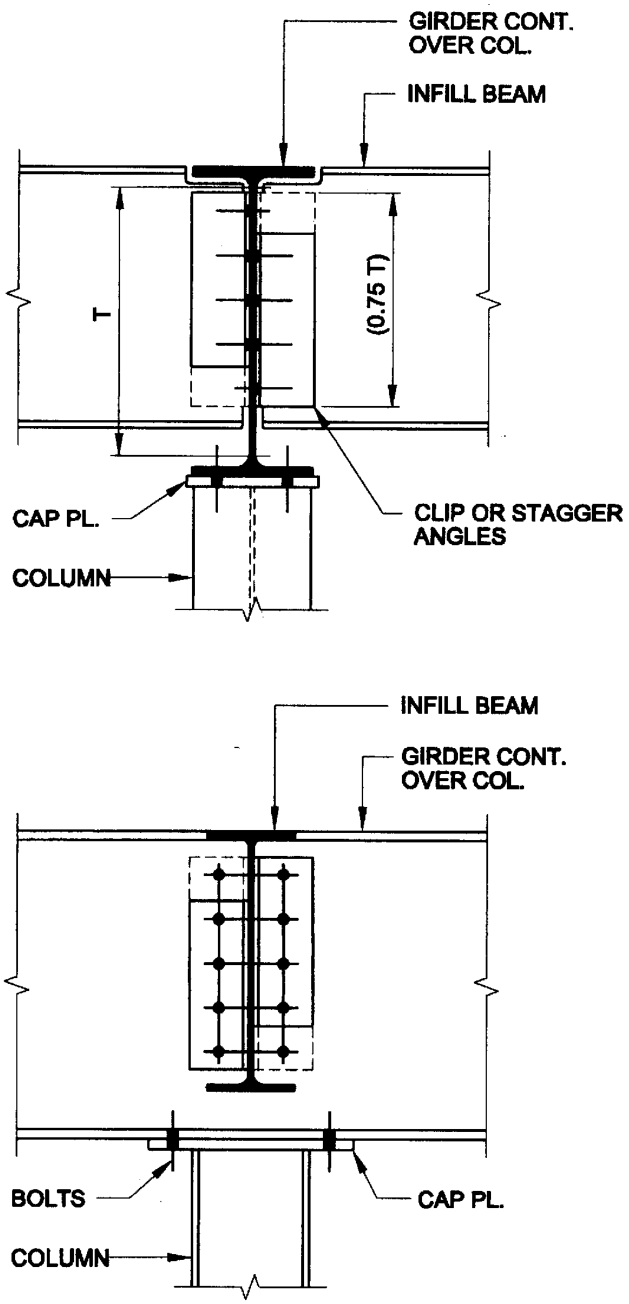


Figure 2–2a. Beam framing continuously over column top, stability provided with connections of infill beams.

1. When an infill beam frames into the continuous beam at the column top, the required stability normally can be provided by using connection element(s) for the infill beam that cover three-quarters or more of the T-dimension of the continuous beam. Alternatively, connection elements that cover less than three-quarters of the T-dimension of the continuous beam can be used in conjunction with partial-depth stiffeners in the beam web along with a moment connection between the column top and beam bottom to maintain alignment of the beam/column assembly. A cap plate of reasonable proportions and four bolts will normally suffice.

   In either case, note that OSHA requires that, if two framing infill beams share common holes through a column web or the web of a beam that frames continuously over the top of a column[3], the beam erected first must remain attached while connecting the second.

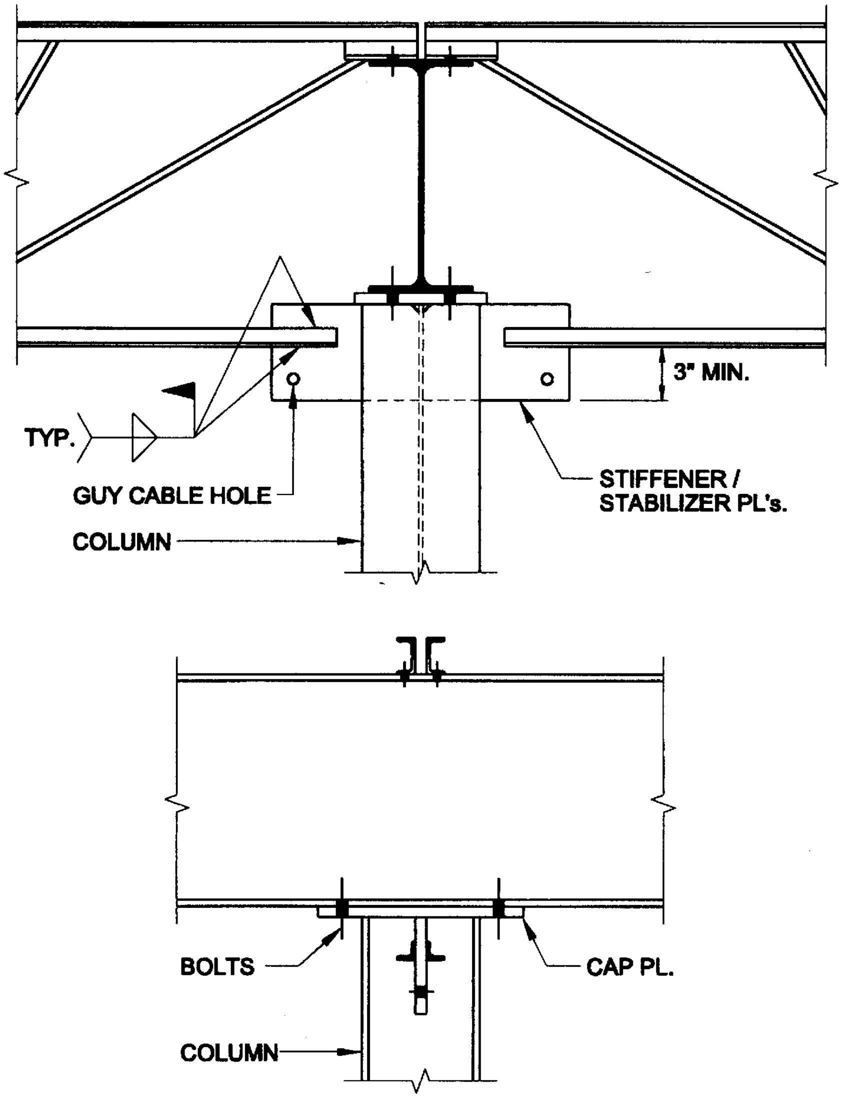


Figure 2–2b. Beam framing continuously over column top, stability provided with welded joist-chord extensions at column top.

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[3] This requirement applies only at the location of the column, not at locations away from the column.

2. When joists frame into the continuous beam or girder, the required stability normally can be provided by using bottom chord extensions connected to the column top. The resulting continuity moments must be reported to the joist supplier for their use in the design of the joists and bridging. Note that the continuous beam must still be checked for the concentrated force due to the column reaction per AISC Specification Section J11.

   The position of the bottom chord extension relative to the column cap plate will affect the bottom chord connection detail. When the extension aligns with the cap plate, the load path and force transfer is direct. When the extension is below the column cap plate, the column must be designed to stabilize the beam bottom flange and the connection between the extension and the column must develop the continuity/brace force. When the extension is above the column top, the beam web must have the necessary strength and stiffness to adequately brace the beam bottom/column top.

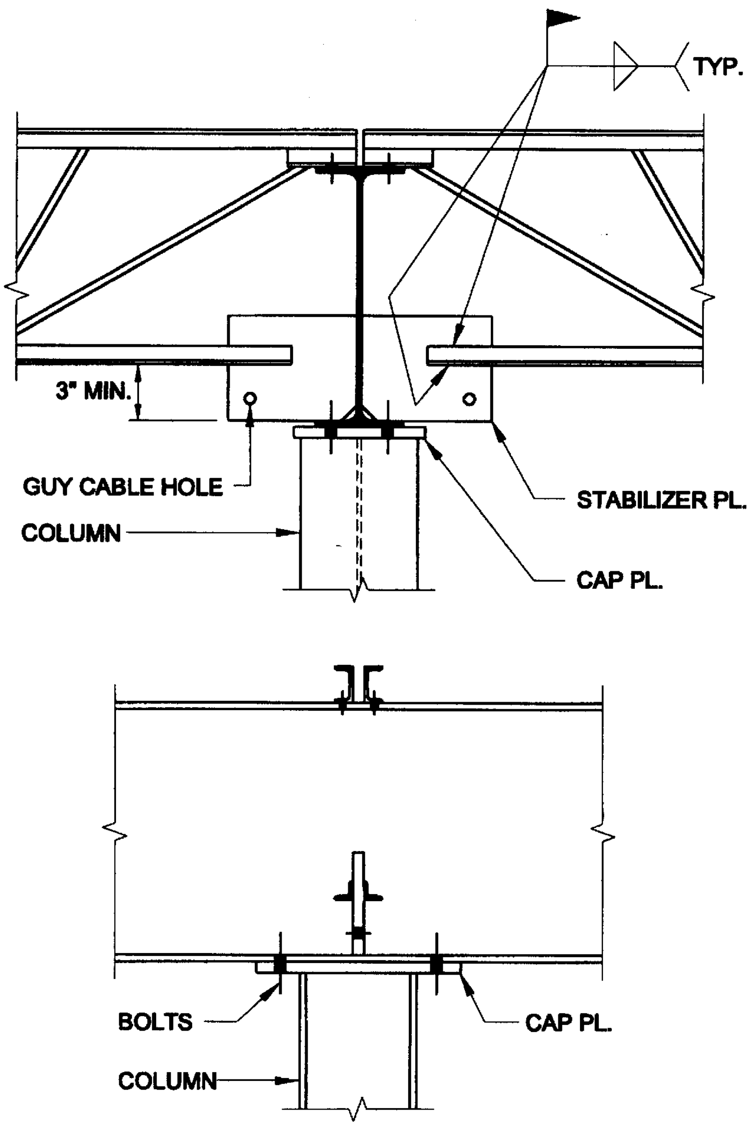


Figure 2–2c. Beam framing continuously over column top, stability provided with welded joist-chord extensions above column top.

3. If connection of the joist bottom chord extensions to the column must be avoided, the required stability can be provided with a diagonal brace that satisfies the strength and stiffness requirements in AISC Specification Appendix 6. Providing a relatively shallow angle with respect to the horizontal can minimize gravity-load effects in the diagonal brace.

   Alternatively, the required stability can be provided with stiffeners in the beam web along with a moment connection between the column top and beam bottom to maintain alignment of the beam/column assembly. A cap plate of reasonable proportions and four bolts will normally suffice.

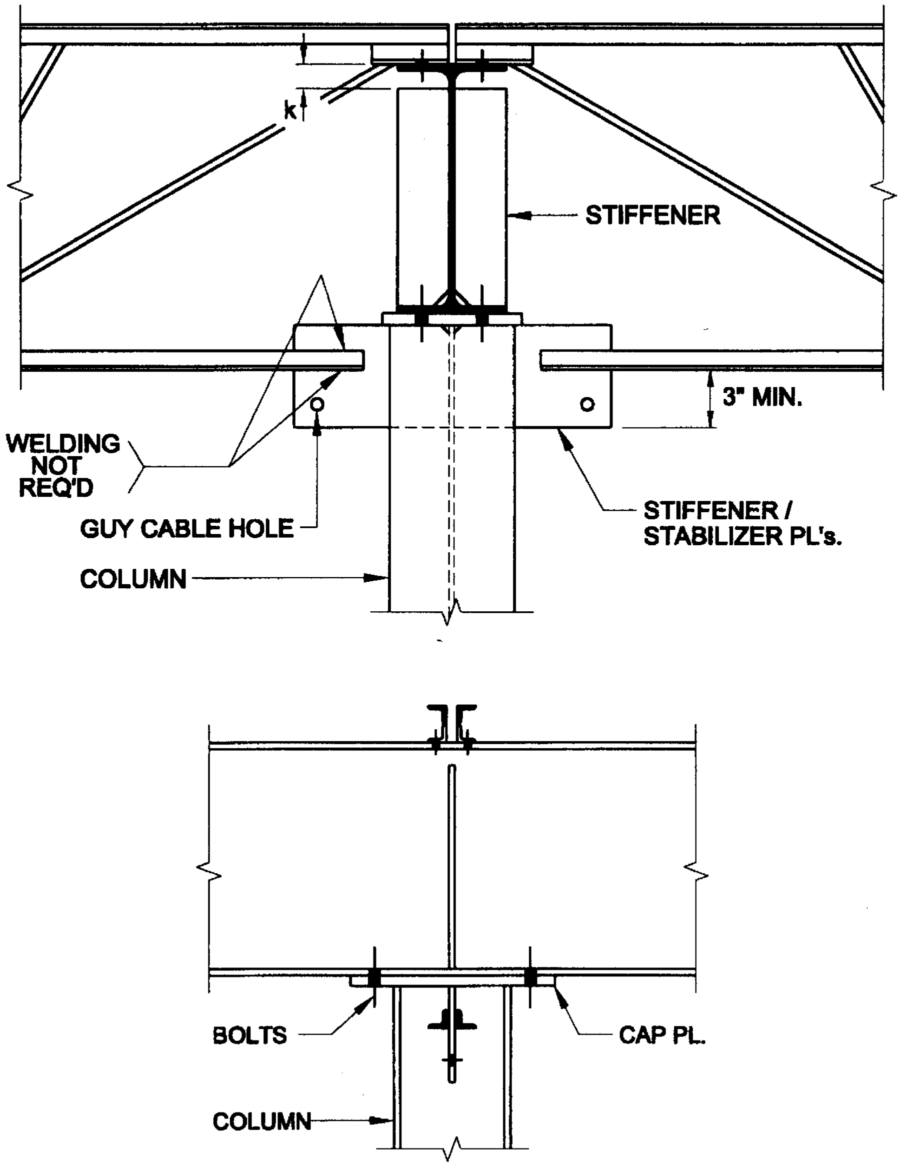


Figure 2-2d. Beam framing continuously over column top, stability provided with transverse stiffeners, joist-chord extensions located at column top not welded.

In atypical framing situations, such as when very deep girders are used, the strength and stiffness requirements in AISC Specification Appendix 6 can be applied for both the beam and the column to ensure the stability of the assembly. It may also be possible to demonstrate in a limited number of cases, such as with continuous beams with thick webs and relatively shallow depths, that the column and beam have been properly designed without providing infill beam connections, connected joist extensions, stiffeners, or diagonal braces as described above. In this case, a properly designed moment connection is still required between the beam bottom flange and the column top. In any case, it should be noted that the assembly must also meet the requirements in AISC Specification Section J11.

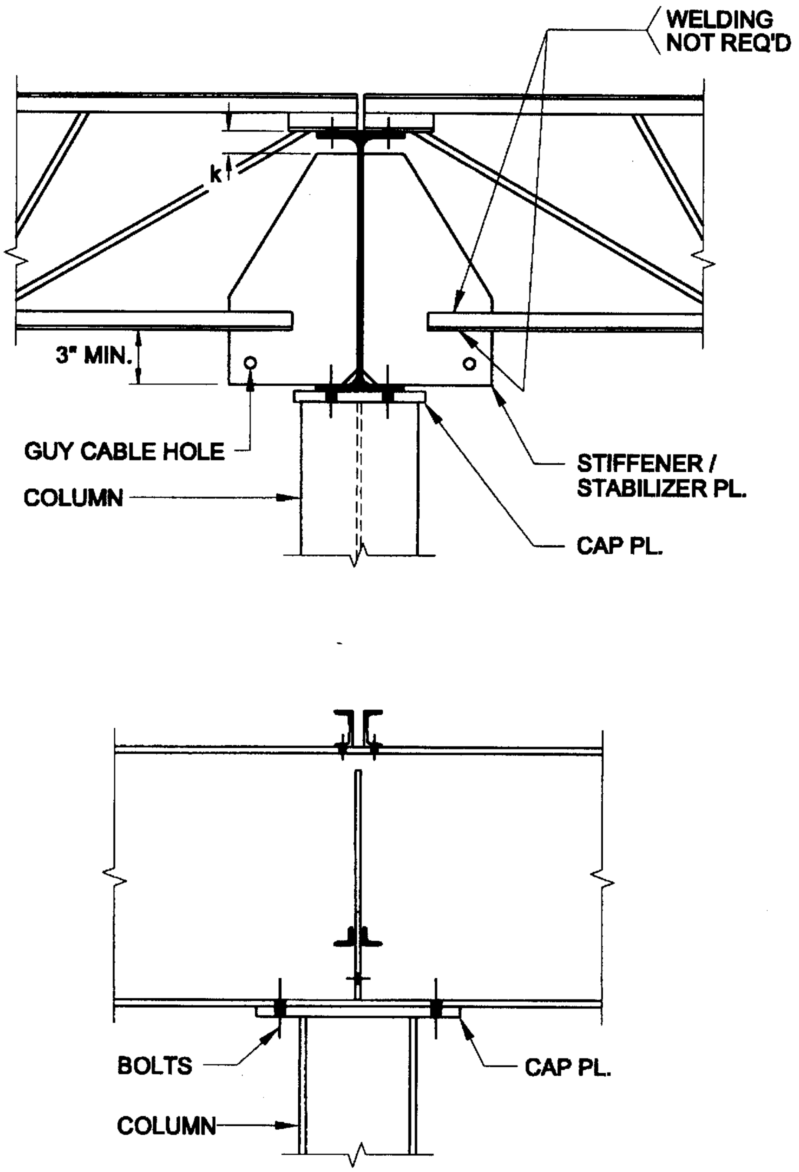


Figure 2–2e. Beam framing continuously over column top, stability provided with stiffener plates, joist-chord extensions located above column top not welded.

PROPERLY SPECIFYING MATERIALS

Availability

The general availability of structural shapes, HSS, and pipe is determined by an annual AISC survey of producers and summarized in AISC's *Modern Steel Construction* magazine. The availability summary for W-, M-, S-, and HP-shapes, channels, and angles is published in the January issue. The availability summary for HSS and pipe is published in the July issue. This information is also available at www.aisc.org.

Material Specifications

Applicable material specifications are as shown in the following tables:

- Structural shapes in Table 2–3.
- Plate and bar products in Table 2–4.
- Fastening products in Table 2–5.

Preferred material specifications are indicated in black shading. Other applicable material specifications are as shown in grey shading. The availability of grades other than the preferred material specification should be confirmed prior to their specification.

Cross-sectional dimensions and production tolerances are addressed as indicated under "Standard Mill Practices" in Part 1.

Other Products

Raised-Pattern Floor Plates

ASTM A786 is the standard specification for rolled steel floor plates. As floor-plate design is seldom controlled by strength considerations, ASTM A786 "commercial grade" is commonly specified. If so, per ASTM A786 Section 5.1.2, "the product will be supplied 0.33 percent maximum carbon and without specified mechanical properties." Alternatively, if a defined strength level is desired, ASTM A786 raised-pattern floor plate can be ordered to a defined plate specification, such as ASTM A36, A572, or A588; see ASTM A786 Sections 5.1.2, Section 8, and Appendix Table X1.1.

Sheet and Strip

Sheet and strip products, which are generally thinner than structural plate and bar products (see Table 2–2), are produced to such ASTM specifications as A570, A606, or A607.

Filler Metal

The appropriate filler metal for structural steel is as summarized in ANSI/AWS D1.1-2004 Table 3.1 for the various combinations of base metal specification, and grade and electrode specification. Weld strengths in this Manual are based upon a tensile strength level of 70 ksi.

Shear-Stud Connectors

As specified in ANSI/AWS D1.1 Chapter 7 (Section 7.2.6 and Table 7.1), Type B shear stud connectors made from ASTM A108 material are used for the interconnection of steel and concrete elements in composite construction ($F_u = 65$ ksi).

Open-Web Steel Joists

The AISC Code of Standard Practice does not include steel joists in its definition of structural steel. Steel joists are designed and fabricated per the requirements of specifications published by the Steel Joist Institute. Refer to SJI literature for further information.

Castellated Beams

Castellated beams, also known as cellular beams, are members constructed by cutting along a staggered pattern down the web of a wide-flange member, offsetting the resulting pieces such that the deepest points of the cut are in contact, and welding the two pieces together, thereby creating a member with holes along its web. Castellated beams are currently designed and fabricated as a proprietary product. For more information, contact the manufacturer.

Steel Castings and Forgings

Steel castings are specified as ASTM A27 grade 65-35 or ASTM A148 grade 80-35. Steel forgings are specified as ASTM A668.

Forged Steel Structural Hardware

Forged steel structural hardware products, such as clevises, turnbuckles, eye nuts, and sleeve nuts, are occasionally used in building design and construction. These products are generally forged according to ASTM A668 Class A requirements. ASTM A29, grade 1035 material is commonly used in the manufacture of clevises and turnbuckles. ASTM A29, grade 1030 material is commonly used in the manufacture of steel eye nuts and steel eye bolts. ASTM A29 grade 1018 material is commonly used in the manufacture of sleeve nuts. Other products, such as steel rod ends, steel yoke ends and pins, cotter pins, and coupling nuts are commonly provided generically as "carbon steel."

The dimensional and strength characteristics of these devices are fully described in the literature provided by their manufacturer. Note that manufacturers usually provide strength characteristics in terms of a "safe working load" with a safety factor as high as 5, assuming that the product will be used in rigging or similar applications subject to dynamic loading. The manufacturer's safe working load may be overly conservative for permanent installations and similar applications subject to static loading only.

If desired, the published safe working load can be converted into an available strength with reliability consistent with that of other statically loaded structural materials. In this case, the nominal strength, R_n, is determined as:

$$R_n = (\textit{safe working load}) \times (\textit{manufacturer's safety factor})$$

and the available strength, ϕR_n or R_n/Ω, is determined using

$$\phi = 0.50 \text{ (LRFD)} \qquad \Omega = 3.00 \text{ (ASD)}$$

Crane Rails

Crane rails are furnished to ASTM A759, ASTM A1 and/or manufacturer's specifications and tolerances.

Most manufacturers chamfer the top and sides of the crane-rail head at the ends, unless specified otherwise, to reduce chipping of the running surfaces. Often, crane rails are ordered as end-hardened, which improves the resistance of the crane-rail ends to impact that occurs as the moving wheel contacts it during crane operation. Alternatively, the entire rail can be ordered as heat-treated. When maximum wheel loading or controlled cooling is needed, refer to manufacturers' catalogs. Purchase orders for crane rails should be noted "for crane service."

Light 40-lb rails are available in 30-ft lengths, 60-lb rails in 30-, 33-, or 39-ft lengths, standard rails in 33- or 39-ft lengths, and crane rails up to 80 ft. Consult manufacturer for availability of other lengths. Rails should be arranged so that joints on opposite sides of the crane runway will be staggered with respect to each other and with due consideration to the wheelbase of the crane. Rail joints should not occur at crane girder splices. Odd lengths that must be included to complete a run or obtain the necessary stagger should be not less than 10 ft long. Rails are furnished with standard drilling in both standard and odd lengths, unless stipulated otherwise on the order.

CONTRACT DOCUMENT INFORMATION

Design Drawings, Specifications, and Other Contract Documents

CASE Document 962D *A Guideline Addressing Coordination and Completeness of Structural Construction Documents*, (Council of American Structural Engineers, American Council of Engineering Companies, 2003) provides comprehensive guidance on the preparation of structural design drawings.

Most provisions in the AISC Specification, RCSC Specification, AWS D1.1, and the Code of Standard Practice are written in mandatory language. Some provisions require the communication of information in the contract documents, some provisions are invoked only when specified in the contract documents, and some provisions require the approval of the owner's designated representative for design if they are to be used. Following is a summary of these provisions in the AISC Specification, RCSC Specification, and Code of Standard Practice:

Required Information

The following communication of information is required in the contract documents:

1. Required drawing information, per Code of Standard Practice Sections 3.1 and 3.1.1 through 3.1.6. and RCSC Specification Section 1.4 (bolting products and joint type).
2. Drawing numbers and revision numbers, per Code of Standard Practice Section 3.5.
3. Structural system description, per Code of Standard Practice Section 7.10.1.
4. Installation schedule for non-structural steel elements in the structural system, per Code of Standard Practice Section 7.10.2.
5. Project schedule, per Code of Standard Practice Section 9.5.1.

Information Required Only When Specified

The following provisions are invoked only when specified in the contract documents:

1. Special material notch-toughness requirements, per AISC Specification Section A3.1c and Section A3.1d.
2. Special connections requiring pretension, per AISC Specification Section J1.10.

3. Bolted joint requirements, per AISC Specification Section J3.1 and RCSC Specification Section 1.4.
4. Special cambering considerations, per AISC Specification Section L2.
5. Special contours and finishing requirements for thermal cutting, per AISC Specification Sections M2.2 and M2.3, respectively.
6. Corrosion protection requirements, if any, per AISC Specification Sections M3.1, M3.2, and M3.5, and Code of Standard Practice Sections 6.5, 6.5.2, and 6.5.3.
7. Responsibility for field touch-up painting, if painting is specified, per AISC Specification Section M4.6 and Code of Standard Practice Section 6.5.4.
8. Special quality assurance and inspection requirements, per AISC Specification Sections M5 and M5.3, and Code of Standard Practice Sections 8.1.3, 8.2, and 8.3.
9. Evaluation procedures, per AISC Specification Section B6.
10. Fatigue requirements, if any, per AISC Specification Section B3.9.
11. Modifications, if any, to the Code of Standard Practice, per Code of Standard Practice Section 1.1.
12. Submittal schedule for shop and erection drawings, per Code of Standard Practice Section 4.2.
13. Mill order timing, special mill testing, and special mill tolerances, per Code of Standard Practice Sections 5.1, 5.2, and 5.2, respectively.
14. Removal of backing bars and run-off-tabs, per Code of Standard Practice Section 6.3.2
15. Special erection mark requirements, per Code of Standard Practice Section 6.6.1.
16. Special delivery and erection sequences, per Code of Standard Practice Sections 6.7.1 and 7.1, respectively.
17. Special field splice requirements, per Code of Standard Practice Section 6.7.4.
18. Specials loads to be considered during erection, per Code of Standard Practice Section 7.10.3.
19. Special safety protection treatments, per Code of Standard Practice Section 7.11.1.
20. Identification of adjustable items, per Code of Standard Practice Section 7.13.1.3.
21. Cuts, alterations, and holes for other trades, per Code of Standard Practice Section 7.15.
22. Revisions to the contract, per Code of Standard Practice Section 9.3
23. Special terms of payment, per Code of Standard Practice Section 9.6.
24. Identification of architecturally exposed structural steel, per Code of Standard Practice Section 10.

Approvals Required

The following provisions require the approval of the owner's designated representative for design, if they are to be used:

1. Bolted-joint-related approvals per RCSC Commentary Section 1.4.
2. Use of electronic or other copies of the design drawings by the fabricator, per Code of Standard Practice Section 4.3.
3. Use of stock materials not conforming to specified ASTM specification, per Code of Standard Practice Section 5.2.3.
4. Correction of errors, per Code of Standard Practice Section 7.14.
5. Inspector-recommended deviations from contract documents, per Code of Standard Practice Section 8.5.6.
6. Contract price adjustment, per Code of Standard Practice Section 9.4.2.

Establishing Criteria for Connections

Code of Standard Practice Section 3.1.2 provides two methods for the establishment of connection criteria.

In the first, the complete design of all connections is shown in the structural design drawings. In this case, Code of Standard Practice Commentary Section 3.1.2 provides a summary of the information that must be included in the structural design drawings.

This method has the advantage that there is no need to provide connection loads, since the connections are completely designed in the structural design drawings. Additionally, it favors greater accuracy in the bidding process, since the connections are fully described in the contract documents.

In the second, the fabricator is allowed to select or complete the connections while preparing the shop and erection drawings, using the information provided by the owner's designated representative for design per Code of Standard Practice Section 3.1.2. In this case, Code of Standard Practice Commentary Section 3.1.2 clarifies the intention that connections that can be selected or completed by the fabricator include those for which tables appear in the contract documents or the Manual. Other connections should be shown in detail in the structural design drawings.

This method has the advantage that the fabricator's standard connections normally can be used, which often leads to project economy. However, the loads or other connection design criteria must be provided in the structural design drawings. Design loads and required strengths for connections should be provided in the structural design drawings and the design method used in the design of the frame (ASD or LRFD) must be indicated on the drawings.

In either method, the resulting shop and erection drawings must be submitted to the owner's designated representative for design for review and approval. Following is additional guidance for the communication of connection criteria to the connection designer.

Simple Shear Connections

The full force envelope should be given for each simple shear connection. Because of the potential for overestimation—and underestimation—inherent in approximate methods (Thornton, 1992), actual beam end reactions should be indicated on the design drawings. The most effective method to communicate this information is to place a numeric value at each end of each span in the framing plans.

In the past, beam end reactions were sometimes specified as a percentage of the tabulated uniform load in Manual Part 3. This practice can result in either over- or under-specification of connection reactions and should not be used. The inappropriateness of this practice is illustrated in the following four examples:

1. When beams are selected for serviceability considerations or for shape repetition, the uniform load tables will often result in heavier connections than would be required by the actual design loads.
2. When beams have relatively short spans, the uniform load tables will often result in heavier connections than would be required by the actual design loads.
3. When beams support other framing beams or other concentrated loads occur on girders supporting beams, the end reactions can be higher than 50 percent of the total uniform load.
4. For composite beams, the end reactions can be higher than 50 percent of the total uniform load. The percentage requirement can be increased for this condition, but the resulting approach is still subject to the above considerations.

Moment Connections

The full force envelope should be given for each moment connection. If the owner's designated representative for design can select the governing load combination, its effect alone should be provided. Otherwise, the effects of all appropriate load combinations should be indicated. Additionally, the maximum moment imbalance should also be given for use in the check of panel-zone web shear.

Because of the potential for overestimation—and underestimation—inherent in approximate methods, it is recommended that the actual beam end reactions (moment, shear, and other reactions, if any) be indicated in the structural design drawings. The most effective method to do so may be by tabulation for each joint and load combination.

Although not recommended, beam end reactions can be specified by more general criteria, such as by function of the beam strength. It should be noted, however, that there are several situations in which this approach is not appropriate. For example:

1. When beams are selected for serviceability considerations or for shape repetition, this approach will often result in heavier connections than would be required by the actual design loads.
2. When the column(s) or other members that frame at the joint could not resist the forces and moments determined from the criteria so specified, this approach will often result in heavier connections than would be required by the actual design loads.

In some cases, the structural analysis may require that the actual connections be configured to match the assumptions used in the model. For example, it may be appropriate to release weak-axis moments in a beam-column joint where only strong-axis beam moment strength is required. Such requirements should be indicated in the structural design drawings.

Truss Connections

The full force envelope should be given for each truss-member end connection. If the owner's designated representative for design can select the governing load combination for the entire truss, its effect alone should be provided. Otherwise, the effects of all appropriate load combinations should be indicated in tabular form. This approach will allow a clear understanding of all of the forces on any given joint.

Because of the potential for overestimation—and underestimation—inherent in approximate methods, it is recommended that the actual reactions at the truss member end (axial force and other reactions, if any) be indicated in the structural design drawings. It is also recommended that transfer forces, if any, be so indicated. The most effective method to do so may be by tabulation for each truss member end and load combination.

Although not recommended, truss member end reactions can be specified by more general criteria, such as by maximum member forces (tension or compression) or as a function of the member strength. It should be noted, however, that there are several situations in which such approaches are not appropriate. For example:

1. The specification of maximum member forces does not permit a check of the member forces at a joint if there are different load combinations governing the member designs at that joint. Nor does it reflect the possibility of load reversal as it may influence the design.
2. The specification of a percentage of member strength may not properly account for the interaction of forces at a joint or the transfer force through the joint. Additionally, it may not allow for a cross-check of all forces at a joint.

In either case, this approach will often result in heavier connections than would be required by the actual design loads.

Note that it is not necessary to specify a minimum connection strength as a percent of the member strength as a default. However, when trusses are shop assembled or field assembled on the ground for subsequent erection, consideration should be given to the loads that will be induced during handling, shipping, and erection.

Horizontal and Vertical Bracing Connections

The recommendations for truss connections above also apply in general to bracing connections with the following additional comments.

Bracing connections may involve the interaction of gravity and lateral loads on the frame. In some cases, such as V- and inverted V-bracing (also known as Chevron bracing), gravity loads alone may govern design of the braces and their connections. Thus, clarity in the specification of loads and reactions is critical to properly consider the potential interaction of gravity and lateral loads at floors and roofs.

Strut and Tie Connections

Floor and roof members in braced bays and adjacent bays may function as struts or ties in addition to carrying gravity loads. Therefore the recommendations for simple shear connections and bracing connections above apply in combination.

Column Splices

Column splices may resist moments, shears, and tensions in addition to gravity forces. Typical column splices are discussed in Part 14. As in the case of the other connections discussed above, unless the column splices are fully designed in the Construction Documents, forces and moments for the splice designs should be provided in the Construction Documents. Since column splices are located away from the girder/column joint and moments vary in the height of the column, an accurate assessment of the forces and moments at the column splices will usually significantly reduce their cost and complexity.

TOLERANCES

The effects of mill, fabrication, and erection tolerances all require consideration in the design and construction of structural steel buildings. However, the accumulation of the mill tolerances and fabrication tolerances shall not cause the erection tolerances to be exceeded, per Code of Standard Practice Section 7.12.

Mill Tolerances

Mill tolerances are those variations that could be present in the product as-delivered from the rolling mill. These tolerances are given as follows:

1. For structural shapes and plates, see ASTM A6.
2. For HSS, see ASTM A500 (or other applicable ASTM specification for HSS).
3. For pipe, see ASTM A53.

A summary of standard mill practices is also given in Part 1.

Fabrication Tolerances

Fabrication tolerances are generally provided in AISC Specification Section M2 and Code of Standard Practice Section 6.4. Additional requirements that govern fabrication are as follows:

1. Compression joint fit-up, per AISC Specification Section M4.4.
2. Roughness limits for finished surfaces, per Code of Standard Practice Section 6.2.2.
3. Straightness of projecting elements of connection materials, per Code of Standard Practice Section 6.3.1.
4. Finishing requirements at locations of removal of run-off tabs and similar devices, per Code of Standard Practice Section 6.3.2.

Erection Tolerances

Erection tolerances are generally provided in AISC Specification Section M4 and Code of Standard Practice Section 7.13. Note that the tolerances specified therein are predicated upon the proper installation of the following items by the owner's designated representative for construction:

1. Building lines and benchmarks, per Code of Standard Practice Section 7.4.
2. Anchorage devices, per Code of Standard Practice Section 7.5.
3. Bearing devices, per Code of Standard Practice Section 7.6.
4. Grout, per Code of Standard Practice Section 7.7.

Building Façade Tolerances

The preceding mill, fabrication, and erection tolerances can be maintained with standard equipment and workmanship. However, the accumulated tolerances for the structural steel and the building façade must be accounted for in the design so that the two systems can be properly mated in the field. This is normally accomplished by specifying adjustable connections in the contract documents, per Code of Standard Practice Section 7.13.1.3.

The required adjustability normally can be determined from the building façade tolerances and the accumulation of mill, fabrication, and erection tolerances at the mid-span point of the spandrel beam. The actual locations of the anchor-rod group and column base, the actual slope of the columns, and the actual sweep of the spandrel beam all affect the accumulation of tolerances in the structural steel at this critical location. Even if each of these is properly within the permitted envelope, significant variations will normally occur.

Figures 2–3a, 2–4a, and 2–5a illustrate details that are not recommended because they do not provide for adjustment. Figures 2–3b, 2–4b, and 2–5b illustrate recommended alternative details that do provide for adjustability. Note that diagonal structural and stability bracing elements have been omitted in these details to improve the clarity of presentation regarding adjustability. Also, note that all elements beyond the slab edge are normally not structural steel, per Code of Standard Practice Section 2.2, and are shown for the purposes of illustration only.

The bolted details in Figures 2–4b and 2–5b can be used to provide field adjustability with slotted holes as shown. Further adjustability can be provided in these details, if necessary, by removing the bolts and clamping the connection elements for field-welding. Alternatively, when the slab edge angle or plate in Figure 2–4b is shown as field-welded and identified as adjustable in the contract documents, it can be provided to within a horizontal tolerance of ±3/8 in., per Code of Standard Practice Section 7.13.1.3. However, if the item were not shown as field-welded and identified as adjustable in the contract documents, it would likely be attached in the shop or attached in the field to facilitate the concrete pour and not be suitable to provide for the necessary adjustment.

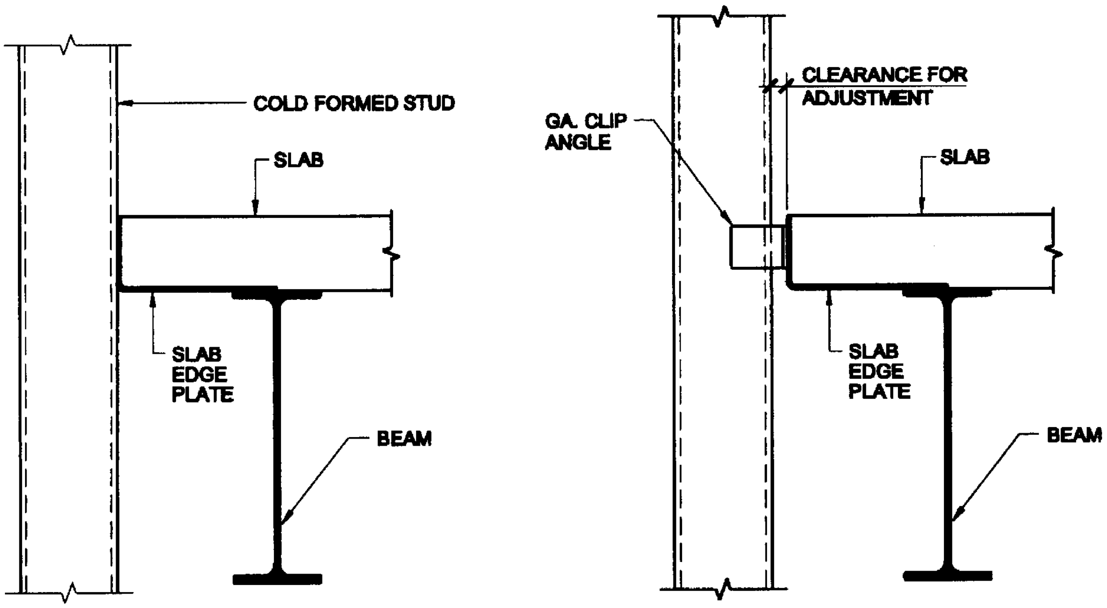
With adjustable connections specified in design and provided in fabrication, the actions taken on the job site will allow for a successful façade installation. Per the Code of Standard Practice definition of established column line (see Code of Standard Practice Glossary), proper placement of this line by the owner's designated representative for construction based upon the actual anchor-rod/column-center locations will assure that all subcontractors are working from the same information. When sufficient adjustment cannot be accommodated within the adjustable connections provided, a common solution is to allow the building façade to deviate (or drift) from the theoretical location to follow the as-built locations of the structural steel framing and concrete floor slabs. A survey of the as-built locations of these elements can be used to adjust the placement of the building façade accordingly. In this case, the adjustable connections can serve to ensure that no abrupt changes occur in the façade.

CAMBER, SWEEP, AND STRAIGHTENING

Beam Camber and Sweep

Camber denotes a curve in the vertical plane. Sweep denotes a curve in the horizontal plane. Camber and sweep are provided in beams, when required, by the fabricator per Code of Standard Practice Section 6.4.4, either by cold bending or by hot bending.

Cambering and sweeping induce residual stresses similar to those that develop in rolled structural shapes, as elements of the shape cool from the rolling temperature at different rates. In general, these residual stresses do not affect the design strength of structural members, since the effect of residual stresses is considered in the provisions of the AISC Specification.



(a) WITHOUT ADJUSTMENT [NOT RECOMMENDED]

(b) WITH ADJUSTMENT [RECOMMENDED]

Figure 2–3. Attaching cold-formed steel façade systems to structural steel framing.

Cold Bending

The inelastic deformations required in common cold-bending operations, such as for beam cambering, normally fall well short of the strain-hardening range. Specific limitations on cold-bending capabilities should be obtained from those that provide the service. However, the following general guidelines may be useful in the absence of other information:

1. The minimum radius for camber induced by cold bending in members up to a nominal depth of 30 in. is between 10 and 14 times the depth of the member. Deeper members may require a larger minimum radius.
2. Cold bending may be used to provide sweep in members to practically any radius desired.
3. A length limit of 40 to 50 ft is practical.

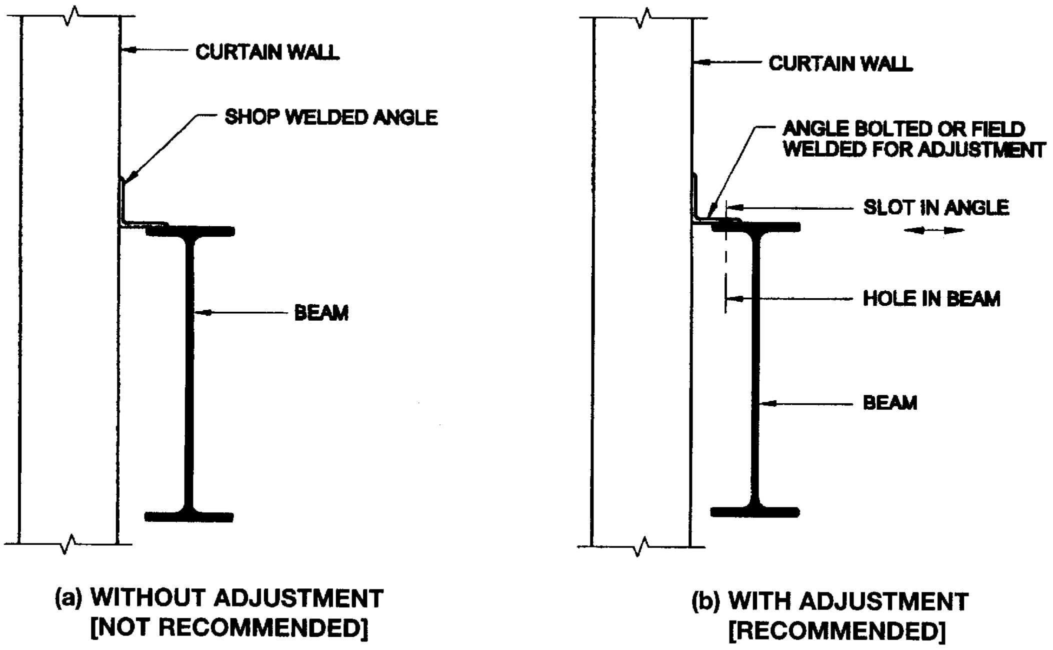
When curvatures and the resulting inelastic deformations are significant and corrective measures are required, the effects of cold work on the strength and ductility of the structural steels largely can be eliminated by thermal stress relief or annealing.

Hot Bending

The controlled application of heat can be used in the shop and field to provide camber or sweep. The member is rapidly heated in selected areas that tend to expand, but are restrained by the adjacent cooler areas, causing inelastic deformations in the heated areas and a change in the shape of the cooled member.

The mechanical properties of steels are largely unaffected by such heating operations, provided the maximum temperature does not exceed the temperature limitations given in AISC Specification Section M2.1. Temperature-indicating crayons or other suitable means should be used during the heating process to ensure proper regulation of the temperature.

Heat curving induces residual stresses that are similar to those that develop in hot-rolled structural shapes as they cool from the rolling temperature because all parts of the shape do



(a) WITHOUT ADJUSTMENT [NOT RECOMMENDED]

(b) WITH ADJUSTMENT [RECOMMENDED]

Figure 2–4. Attaching curtain-wall façade systems to structural steel framing.

not cool at the same rate. The residual stresses from heating operations generally do not affect the design strength of structural members, since the effect of residual stresses is considered in the provisions of the AISC Specification.

Truss Camber

Camber is provided in trusses, when required, by the fabricator per Code of Standard Practice Section 6.4.5, by geometric relocation of panel points and adjustment of member lengths based upon the camber requirements as specified in the contract documents.

Straightening

All structural shapes are straightened at the mill after rolling, either by rotary or gag straightening, to meet the aforementioned mill tolerances. Similar processes and/or the controlled application of heat can be used in the shop or field to straighten a curved or distorted member. These processes are normally applied in a manner similar to those used to induce camber and sweep, as described above.

FIRE PROTECTION AND ENGINEERING

Complete coverage of fire protection and engineering for steel structures is included in AISC Design Guide 19, *Fire Resistance of Structural Steel Framing* (Ruddy et. al., 2003).

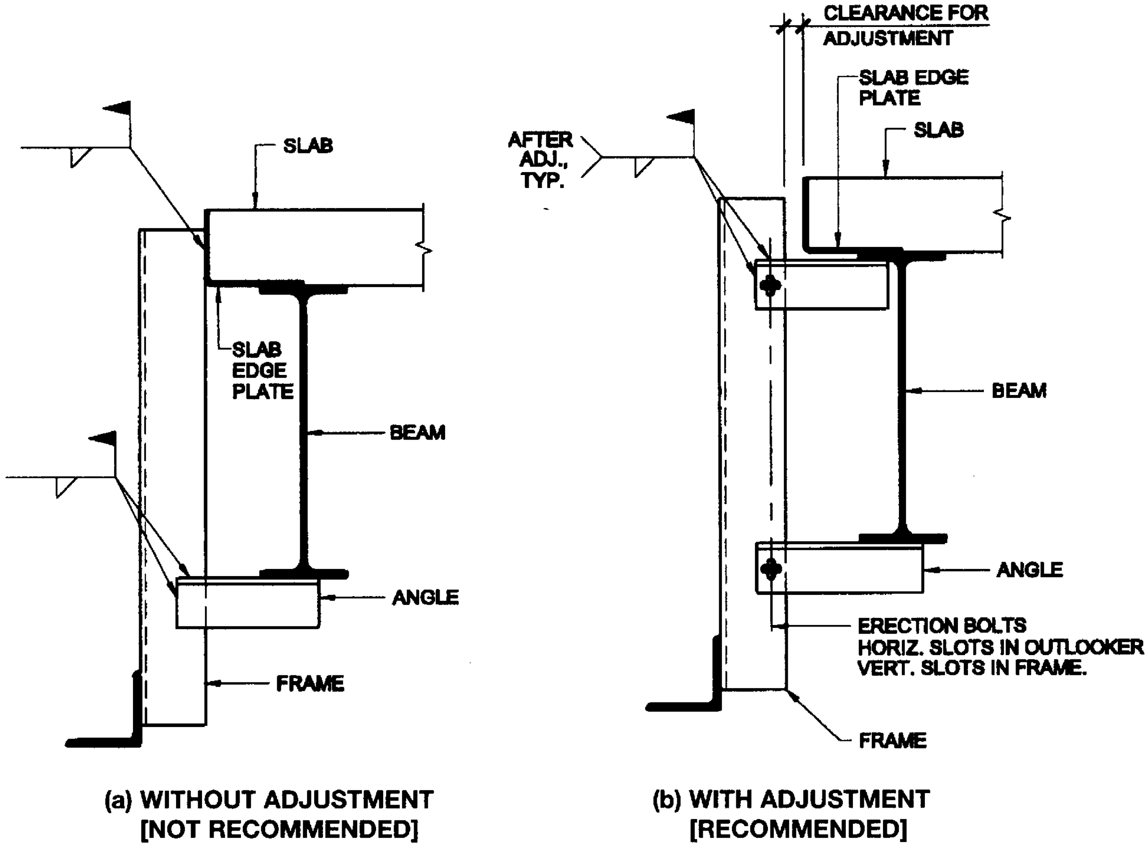


Figure 2–5. Attaching masonry façade systems to structural steel framing.

CORROSION PROTECTION

In building structures, corrosion protection is not required for steel that will be enclosed by building finish, coated with a contact-type fireproofing, or in contact with concrete. When enclosed, the steel is trapped in a controlled environment and the products required for corrosion are quickly exhausted, as indicated in AISC Commentary Section M3. A similar situation exists when steel is fireproofed or in contact with concrete. Accordingly, shop primer or paint is not required unless specified in the contract documents, per AISC Specification Section M3.1. Per Code of Standard Practice Section 6.5, steel that is to remain unpainted need only be cleaned of heavy deposits of oil and grease by appropriate means after fabrication.

Corrosion protection is required, however, in exterior exposed applications. Likewise, steel must be protected from corrosion in aggressively corrosive applications, such as a paper processing plant, a structure with oceanfront exposure, or when temperature changes can cause condensation. Corrosion should also be considered when connecting steel to dissimilar metals. Guidance on steel compatibility with metal fasteners is provided in Table 2–6.

When surface preparation other than the cleaning described above is required, an appropriate SSPC grade of cleaning should be specified in the contract documents. A summary of the SSPC surface preparation specifications (SSPC, 2000) is provided in Table 2–7. SSPC SP 2 is the normal grade of cleaning when cleaning is required.

For further information, refer to the publications of SSPC: The Society for Protective Coatings, the American Galvanizers Association (AGA), and the National Association of Corrosion Engineers International (NACE).

RENOVATION AND RETROFIT OF EXISTING STRUCTURES

The provisions in AISC Specification Section B6 govern the evaluation of existing structures. Historical data on available steel grades and hot-rolled structural shapes, including dimensions and properties, is available in AISC Design Guide 15 *Rehabilitation and Retrofit Guide* (Brockenbrough, 2002), and the companion database of historic shape properties from 1873-1999, titled *AISC Search Utility for Structural Steel Shapes* (AISC, 2003). See also Ricker (1988) and Tide (1990).

THERMAL EFFECTS

Expansion and Contraction

The average coefficient of expansion e for structural steel between 70 and 100 degrees F is 0.0000065 for each degree F. This value is a reasonable approximation of the coefficient of thermal expansion for temperatures less than 70 degrees F. For temperatures from 100 to 1,200 degrees F, the change in length per unit length per degree F, ε, is:

$$\varepsilon = (6.1 + 0.0019t)10^{-6}$$

where t is the initial temperature in degrees F. The coefficients of expansion for other building materials can be found in Table 17–11.

Although buildings are typically constructed of flexible materials, expansion joints are often required in roofs and the supporting structure when horizontal dimensions are large. The maximum distance between expansion joints is dependent upon many variables, including ambient temperature during construction and the expected temperature range during the lifetime of the building.

Figure 2–6 (Federal Construction Council, 1974) provides guidance based on design temperature change for maximum spacing of structural expansion joints in beam-and-column-framed buildings with pinned column bases and heated interiors. The report includes data for numerous cities and gives five modification factors to be applied as appropriate:

1. If the building will be heated only and will have pinned column bases, use the maximum spacing as specified;
2. If the building will be air-conditioned as well as heated, increase the maximum spacing by 15 percent, provided the environmental control system will run continuously;
3. If the building will be unheated, decrease the maximum spacing by 33 percent;
4. If the building will have fixed column bases, decrease the maximum spacing by 15 percent;
5. If the building will have substantially greater stiffness against lateral displacement in one of the plan dimensions, decrease the maximum spacing by 25 percent.

When more than one of these design conditions prevail in a building, the percentile factor to be applied is the algebraic sum of the adjustment factors of all the various applicable conditions. Most building codes include restrictions on location and maximum spacing of fire walls, which often become default locations for expansion joints.

The most effective expansion joint is a double line of columns that provides a complete and positive separation. Alternatively, low-friction sliding elements can be used. Such systems, however, are seldom totally friction-free and will induce some level of inherent restraint to movement.

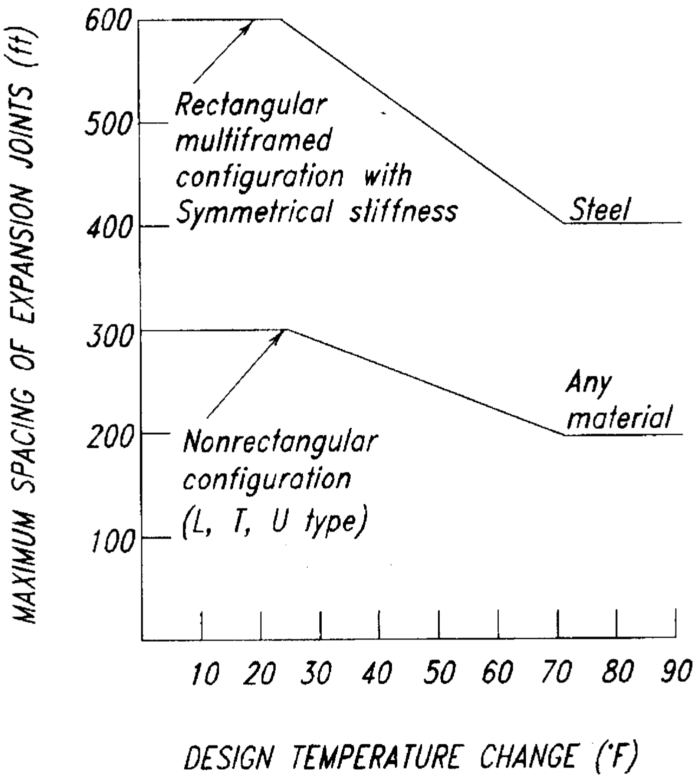


Figure 2–6. Recommended maximum expansion-joint spacing.

Elevated-Temperature Service

For applications involving short-duration loading at elevated temperature, the variations in yield strength, tensile strength, and modulus of elasticity are given AISC Design Guide 19. For applications involving long-duration loading at elevated temperatures, the effects of creep must also be considered. For further information, see Brockenbrough and Merritt (1999; pp. 1.20-1.22).

FATIGUE AND FRACTURE CONTROL

Avoiding Brittle Fracture

By definition, brittle fracture occurs by cleavage at a stress level below the yield strength. Generally, a brittle fracture can occur when there is a sufficiently adverse combination of tensile stress, temperature, strain rate, and geometrical discontinuity (notch). The exact combination of these conditions and other factors that will cause brittle fracture cannot be readily calculated. Consequently, the best guide in selecting steel material that is appropriate for a given application is experience.

The steels listed in AISC Specification Section A3.1a, Section A3.1c, and Section A3.1d have been successfully used in a great number of applications, including buildings, bridges, transmission towers, and transportation equipment, even at the lowest atmospheric temperatures encountered in the United States. Nonetheless, it is desirable to minimize the conditions that tend to cause brittle fracture: triaxial state-of-stress, increased strain rate, strain aging, stress risers, welding residual stresses, areas of reduced notch toughness, and low-temperature service.

1. Triaxial state-of-stress: While shear stresses are always present in a uniaxial or biaxial state-of-stress, the maximum shear stress approaches zero as the principal stresses approach a common value in a triaxial state-of-stress. A triaxial state-of-stress can also result from uniaxial loading when notches or geometrical discontinuities are present. A triaxial state-of-stress will cause the yield stress of the material to increase above its nominal value, resulting in brittle fracture by cleavage, rather than ductile shear deformations. As a result, in the absence of critical-size notches, the maximum stress is limited by the yield stress of the nearby unaffected material. Triaxial stress conditions should be avoided, when possible.
2. Increased strain rate: Gravity loads, wind loads, and seismic loads have essentially similar strain rates. Impact loads, such as those associated with heavy cranes, and blast loads normally have increased strain rates, which tend to increase the possibility of brittle fracture. Note, however, that a rapid strain rate or impact load is not a required condition for the occurrence of brittle fracture.
3. Strain aging: Cold-working of steel and the strain aging that normally results generally increases the likelihood of brittle fracture, usually due to a reduction in ductility and notch toughness. The effects of cold-work and strain aging can be minimized by selecting a generous forming radius to eliminate or minimize strain hardening.
4. Stress risers: Fabrication operations, such as flame-cutting and welding, may induce geometric conditions or discontinuities that are crack-like in nature, creating stress risers. Intersecting welds from multiple directions should be avoided with properly sized weld access holes to minimize the interaction of these various stress fields. Such conditions should be avoided, when possible, or removed or repaired when they occur.

5. Welding residual stresses: In the as-welded condition, residual stresses near the yield strength of the material will be present in any weldment. Residual stresses and the possible accompanying distortions can be minimized through controlled welding procedures and fabrication methods, including the proper positioning of the components of the joint prior to welding, the selection of welding sequences that will minimize distortions, the use of preheat as appropriate, the deposition of a minimum volume of weld metal with a minimum number of passes for the design condition, and proper control of interpass temperatures and cooling rates. In fracture-sensitive applications, notch toughness should be specified for both the base metal and the filler metal.
6. Areas of reduced notch toughness: Such areas can be found in the core areas of heavy shapes and plates and the k-Area of rotary-straightened W-shapes. Accordingly, AISC Specification Sections A3.1c and Section A3.1d include special requirements for material notch toughness.
7. Low-temperature service: While steel yield strength, tensile strength, modulus of elasticity, and fatigue strength increase as temperature decreases, ductility and toughness decrease. Furthermore, there is a temperature below which steel subjected to tensile stress may fracture by cleavage, with little or no plastic deformation, rather than by shear, which is usually preceded by considerable inelastic deformation. Note that cleavage and shear are used in the metallurgical sense to denote different fracture mechanisms.

When notch toughness is important, Charpy V-notch testing can be specified to ensure a certain level of energy absorption at a given temperature, such as 15 ft-lbs at 70 degrees F. Note that the appropriate test temperature may be higher than the lowest operating temperature depending upon the rate of loading. Although it is primarily intended for bridge-related applications, the information in ASTM A709 Section S83 (including Tables S1.1, S1.2, and S1.3) may be useful in determining the proper level of notch toughness that should be specified.

In many cases, weld metal notch toughness exceeds that of the base metal. Filler metals can be selected to meet a desired minimum notch toughness value. For each welding process, electrodes exist that have no specified notch toughness requirements. Such electrodes should not be assumed to possess any minimum notch toughness value. When notch toughness is necessary for a given application, the desired value or an appropriate electrode should be specified in the contract documents.

For further information, refer to Fisher et al. (1998), Barsom and Rolfe (1999), and Rolfe (1977).

Avoiding Lamellar Tearing

Although lamellar tearing is less common today, the restraint against solidified weld deposit contraction inherent in some joint configurations can impose a tensile strain high enough to cause separation or tearing on planes parallel to the rolled surface of the element being joined. The incidence of this phenomenon can be reduced or eliminated through greater understanding by designers, detailers, and fabricators of the inherent directionality of rolled steel, the importance of strains associated with solidified weld deposit contraction in the presence of high restraint (rather than externally applied design forces), and the need to adopt appropriate joint and welding details and procedures with proper weld metal for through-thickness connections.

Research by Melendrez and Dexter (Dexter and Melendrez, 2000) demonstrates that W-shapes are not susceptible to lamellar tearing or other through-thickness failures when welded tee joints are made to the flanges at locations away from member ends. When needed for other conditions, special production practices can be specified for steel plates to assist in reducing the incidence of lamellar tearing by enhancing through-thickness ductility. For further information, refer to ASTM A770. However, it must be recognized that it is more important and effective to properly design, detail, and fabricate to avoid highly restrained joints. AISC (1973) provides guidelines that minimize potential problems.

WIND AND SEISMIC DESIGN

In general, nearly all building design and construction can be classified into one of two categories: wind and low-seismic applications, and high-seismic applications.

Wind and Low-Seismic Applications

Wind and low-seismic applications are those in which the seismic response modification factor R may be taken as equal to or less than 3 for design purposes. Such buildings are designed to meet the provisions in the AISC Specification based upon the code-specified forces distributed throughout the framing, assuming a nominally elastic structural response. The resulting systems have normal levels of ductility.

The seismic response modification factor, R, essentially represents the ratio of the forces that would develop under the specified ground motion if the structure had an entirely linearly elastic response to the prescribed design forces (BSSC, 2001).

High-Seismic Applications

High-seismic applications are those in which R is taken greater than 3, and the building is designed to meet the provisions in both the Seismic Provisions and AISC Specification. Note that it does not matter if wind or earthquake controls in this case. The use of R greater than 3 in the calculation of the seismic base shear requires the use of a seismically detailed system that is compatible with R even if wind effects control. High-seismic design and construction will generally cost more than wind and low-seismic design and construction, as the resulting systems are designed to have high levels of ductility.

High-seismic lateral framing systems are configured to be capable of withstanding strong ground motions as they undergo controlled ductile deformations to dissipate energy. Consider the following three examples:

1. Special Concentrically Braced Frames (SCBF)—SCBF are generally configured so that any inelasticity will occur by tension yielding and/or compression buckling in the braces. The connections of the braces to the columns and beams, and between the columns and beams themselves must then be proportioned to remain nominally elastic, as they undergo these deformations.
2. Eccentrically Braced Frames (EBF)—EBF are generally configured so that any inelasticity will occur by shear yielding and/or flexural yielding in the link. The beam outside the link, connections, braces, and columns must then be proportioned to remain nominally elastic, as they undergo these deformations.

3. Special Moment Frames (SMF)—SMF are generally configured so that any inelasticity will occur by flexural yielding in the girders near, but away from, the connection of the girders to the columns. The connections of the girders to the columns and the columns themselves must then be proportioned to remain nominally elastic as they undergo these deformations. Intermediate Moment Frames (IMF) and Ordinary Moment Frames (OMF) are also configured to provide improved seismic performance, although successively lower than that for SMF.

The code-specified base accelerations used to calculate the seismic forces are not necessarily maximums, but rather, they represent the intensity of ground motions that have been selected by the code-writing authorities as reasonable for design purposes. Accordingly, the requirements in both the Seismic Provisions and the AISC Specification must be met so that the resulting frames can then undergo controlled deformations in a ductile, well-distributed manner.

The design provisions for high-seismic systems are also intended to result in distributed deformations throughout the frame, rather than the formation of story mechanisms, so as to increase the level of available energy dissipation and corresponding level of ground motion that can be withstood.

The member sizes in high-seismic frames will be larger than those in wind and low-seismic frames. The connections will also be much more robust so they can transmit the member-strength-driven force demands. Net sections will often require special attention so as to avoid having fracture limit-states control. Special material requirements, design considerations, and construction practices must be followed. For further information on the design and construction of high-seismic systems, see the Seismic Provisions, which are available from the American Institute of Steel Construction, Inc. at www.aisc.org.

Table 2-1
Summary Comparison of Methods for Stability Analysis and Design

	Direct Analysis Method	Effective Length Method	First-Order Analysis Method
Limitations on Use[1]	None	$\Delta_{2nd}/\Delta_{1st} \leq 1.5$	$\Delta_{2nd}/\Delta_{1st} \leq 1.5$ $\alpha P_r/P_y \leq 0.5$
Analysis Type	Second-order elastic[2]		First-order elastic
Geometry of Structure	All three methods use the undeformed geometry in the analysis.		
Minimum or Additional Lateral Loads Required in the Analysis	Minimum[3]; 0.2% of the story gravity load	Minimum; 0.2% of the story gravity load	Additive; at least 0.42% of the story gravity load
Member Stiffnesses Used in the Analysis	Reduced *EA* and *EI*	Nominal *EA* and *EI*	
Design of Columns	$K = 1$ for all frames	$K = 1$ for braced frames. For moment frames, determine K from sidesway buckling analysis[4]	$K = 1$ for all frames[5]
Specification Reference for Method	Appendix 7	Section C2.2a	Section C2.2b

[1] $\Delta_{2nd}/\Delta_{1st}$ is the ratio of second-order drift to first-order drift, which can be taken to be equal to B_2 calculated per Section C2.1b. $\Delta_{2nd}/\Delta_{1st}$ is determined using LRFD load combinations or a multiple of 1.6 times ASD load combinations.

[2] Either a general second-order analysis method or second-order analysis by amplified first-order analysis (the "B_1-B_2 method" described in Section C2.1b) can be used.

[3] This notional load is additive if $\Delta_{2nd}/\Delta_{1st} > 1.5$.

[4] K=1 is permitted for moment frames when $\Delta_{2nd}/\Delta_{1st} \leq 1.1$.

[5] An additional amplification for member curvature effects is required for columns in moment frames.

Table 2–2
AISI Standard Nomenclature for Flat-Rolled Carbon Steel

Thickness, in.	Width, in.					
	To 3½ incl.	Over 3½ To 6	Over 6 To 8	Over 8 To 12	Over 12 To 48	Over 48
0.2300 & thicker	Bar	Bar	Bar	Plate	Plate	Plate
0.2299 to 0.2031	Bar	Bar	Strip	Strip	Sheet	Plate
0.2030 to 0.1800	Strip	Strip	Strip	Strip	Sheet	Plate
0.1799 to 0.0449	Strip	Strip	Strip	Strip	Sheet	Sheet
0.0448 to 0.0344	Strip	Strip	Hot-rolled sheet and strip not generally produced in these widths and thicknesses			
0.0343 to 0.0255	Strip					
0.0254 & thinner						

Table 2-3

Applicable ASTM Specifications for Various Structural Shapes

Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Applicable Shape Series									
												HSS		
					W	M	S	HP	C	MC	L	Rect.	Round	Pipe
Carbon	A36		36	58-80[b]	▨	■	■	▨	■	■	■			
	A53 Gr. B		35	60										■
	A500	Gr. B	42	58									■	
			46	58								■		
		Gr. C	46	62									▨	
			50	62								▨		
	A501		36	58								▨	▨	
	A529[c]	Gr. 50	50	65-100	▨	▨	▨	▨	▨	▨	▨			
		Gr. 55	55	70-100	▨	▨	▨	▨	▨	▨	▨			
High-Strength Low-Alloy	A572	Gr. 42	42	60	▨	▨	▨	▨	▨	▨	▨			
		Gr. 50	50	65[d]	▨	▨	▨	■	▨	▨	▨			
		Gr. 55	55	70	▨	▨	▨	▨	▨	▨	▨			
		Gr. 60[e]	60	75	▨	▨	▨	▨	▨	▨	▨			
		Gr. 65[e]	65	80	▨	▨	▨	▨	▨	▨	▨			
	A618[f]	Gr. I & II	50[g]	70[g]								▨	▨	
		Gr. III	50	65								▨	▨	
	A913	50	50[h]	60[h]	▨	▨	▨	▨	▨	▨	▨			
		60	60	75	▨	▨	▨	▨	▨	▨	▨			
		65	65	80	▨	▨	▨	▨	▨	▨	▨			
		70	70	90	▨	▨	▨	▨	▨	▨	▨			
	A992		50-65[i]	65[i]	■	▨	▨	▨	▨	▨	▨			
Corrosion Resistant High-Strength Low-Alloy	A242		42[j]	63[j]	▨									
			46[k]	67[k]	▨			▨			▨			
			50[l]	70[l]	▨	▨	▨	▨	▨	▨	▨			
	A588		50	70	▨	▨	▨	▨	▨	▨	▨			
	A847		50	70								▨	▨	

■ = Preferred material specification.
▨ = Other applicable material specification, the availability of which should be confirmed prior to specification.
☐ = Material specification does not apply.

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[a] Minimum unless a range is shown.

[b] For shapes over 426 lb/ft, only the minimum of 58 ksi applies.

[c] For shapes with a flange thickness less than or equal to 1½ in. only. To improve weldability a maximum carbon equivalent can be specified (per ASTM Supplementary Requirement S78). If desired, maximum tensile stress of 90 ksi can be specified (per ASTM Supplementary Requirement S79).

[d] If desired, maximum tensile stress of 70 ksi can be specified (per ASTM Supplementary Requirement S91).

[e] For shapes with a flange thickness less than or equal to 2 in. only.

[f] ASTM A618 can also be specified as corrosion-resistant; see ASTM A618.

[g] Minimum applies for walls nominally ¾-in. thick and under. For wall thicknesses over ¾ in., F_y = 46 ksi and F_u = 67 ksi.

[h] If desired, maximum yield stress of 65 ksi and maximum yield-to-tensile strength ratio of 0.85 can be specified (per ASTM Supplementary Requirement S75).

[i] A maximum yield-to-tensile strength ratio of 0.85 and carbon equivalent formula are included as mandatory in ASTM A992.

[j] For shapes with a flange thickness greater than 2 in. only.

[k] For shapes with a flange thickness greater than 1½ in. and less than or equal to 2 in. only.

[l] For shapes with a flange thickness less than or equal to 1½ in. only.

Table 2–4
Applicable ASTM Specifications for Plates and Bars

Steel Type	ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Plates and Bars									
					to 0.75 incl.	over 0.75 to 1.25	over 1.25 to 1.5	over 1.5 to 2 incl.	over 2 to 2.5 incl.	over 2.5 to 4 incl.	over 4 to 5 incl.	over 5 to 6 incl.	over 6 to 8 incl.	over 8
Carbon	A36		32	58-80										■
			36	58-80	■	■	■	■	■	■	■	■	■	
	A529	Gr. 50	50	70-100	▨	▨[b]	▨[b]	▨[b]	▨[b]					
		Gr. 55	55	70-100	▨	▨[b]	▨[b]							
High-Strength Low-Alloy	A572	Gr. 42	42	60	▨	▨	▨	▨	▨	▨	▨	▨		
		Gr. 50	50	65	▨	▨	▨	▨	▨	▨				
		Gr. 55	55	70	▨	▨	▨	▨						
		Gr. 60	60	75	▨	▨								
		Gr. 65	65	80	▨	▨								
Corrosion Resistant High-Strength Low-Alloy	A242		42	63				▨	▨	▨				
			46	67		▨	▨							
			50	70	▨									
	A588		42	63								▨	▨	
			46	67							▨			
			50	70	▨	▨	▨	▨	▨	▨				
Quenched and Tempered Alloy	A514[c]		90	100-130						▨	▨	▨		
			100	110-130	▨	▨	▨	▨	▨					
Quenched and Tempered Low-Alloy	A852[c]		70	90-110	▨	▨	▨	▨	▨	▨				

■ = Preferred material specification.
▨ = Other applicable material specification, the availability of which should be confirmed prior to specification.
☐ = Material specification does not apply.

a Minimum unless a range is shown.
b Applicable to bars only above 1-in. thickness.
c Available as plates only.

Table 2-5
Applicable ASTM Specifications for Various Types of Structural Fasteners

ASTM Designation		F_y Min. Yield Stress (ksi)	F_u Tensile Stress[a] (ksi)	Diameter Range (in.)	High-Strength Bolts		Common Bolts	Nuts	Washers	Direct-Tension-Indicator Washers	Threaded Rods	Shear Stud Connectors	Anchor Rods		
					Conventional	Twist-Off-Type Tension-Control[d]							Hooked	Headed	Threaded & Nutted
A108		—	65	0.375 to 0.75, incl.								■			
A325[d]		—	105	over 1 to 1.5 incl.	■										
		—	120	0.5 to 1, incl.	■										
A490		—	150	0.5 to 1.5	■										
F1852		—	105	1.125		■									
		—	120	0.5 to 1, incl.		■									
A194 Gr. 2H		—	—	0.25 to 4				▨							
A563		—	—	0.25 to 4				■							
F436[b]		—	—	0.25 to 4					■						
F959		—	—	0.5 to 1.5						■					
A36		36	58-80	to 10							■		▨		▨
A193 Gr. B7[e]		—	100	over 4 to 7							▨		▨		▨
		—	115	over 2.5 to 4							▨		▨		▨
		—	125	2.5 and under							▨		▨		▨
A307	Gr. A	—	60	0.25 to 4			■							▨	
	Gr. C	—	58-80	0.25 to 4							▨		▨		▨
A354 Gr. BD		—	140	2.5 to 4 incl.							▨		▨	▨	▨
		—	150	0.25 to 2.5, incl.							▨		▨	▨	▨
A449		—	90	1.75 to 3 incl.	c						▨		▨	▨	▨
		—	105	1.125 to 1.5, incl.	c						▨		▨	▨	▨
		—	120	0.25 to 1, incl.	c						▨		▨	▨	▨
A572	Gr. 42	42	60	to 6							▨		▨		▨
	Gr. 50	50	65	to 4							▨		▨		▨
	Gr. 55	55	70	to 2							▨		▨		▨
	Gr. 60	60	75	to 1.25							▨		▨		▨
	Gr. 65	65	80	to 1.25							▨		▨		▨
A588		42	63	Over 5 to 8, incl.							▨		▨		▨
		46	67	Over 4 to 5, incl.							▨		▨		▨
		50	70	4 and under							▨		▨		▨
A687		105	150 max.	0.625 to 3							▨		▨		▨
F1554	Gr. 36	36	58-80	0.25 to 4									■	■	■
	Gr. 55	55	75-95	0.25 to 4									▨	▨	▨
	Gr. 105	105	125-150	0.25 to 3									▨	▨	▨

■ = Preferred material specification.
▨ = Other applicable material specification, the availability of which should be confirmed prior to specification.
□ = Material specification does not apply.

— Indicates that a value is not specified in the material specification.

[a] Minimum unless a range is shown or maximum (max.) is indicated.

[b] Special washer requirements may apply per RCSC Specification Table 6.1 for some steel-to-steel bolting applications and per Part 14 for anchor-rod applications.

[c] See AISC Specification Section A3.3 for limitations on use of ASTM A449 bolts.

[d] When atmospheric corrosion resistance is desired, Type 3 can be specified.

[e] For anchor rods with temperature and corrosion resistance characteristics.

Table 2–6

Metal Fastener Compatibility to Resist Corrosion

Fastener Metal Base Metal	Zinc and Galvanized Steel	Aluminum and Aluminum Alloys	Steel and Cast Iron	Brasses, Copper, Bronzes, Monel	Martensitic Stainless Steel (Type 410)	Austenitic Stainless Steel (Type 302/304, 303, 305)
Zinc and Galvanized Steel	A	B	B	C	C	C
Aluminum and Aluminum Alloys	A	A	B	C	Not Recommended	B
Steel and Cast Iron	A, D	A	A	C	C	B
Terne (Lead-Tin) Plated Steel Sheets	A, D, E	A, E	A, E	C	C	B
Brasses, Copper, Bronzes, Monel	A, D, E	A, E	A, E	A	A	B
Ferritic Stainless Steel (Type 430)	A, D, E	A, E	A, E	A	A	A
Austenitic Stainless Steel (Type 302/304)	A, D, E	A, E	A, E	A, E	A	A

KEY

A. The corrosion of the base metal is not increased by the fastener.
B. The corrosion of the base metal is marginally increased by the fastener.
C. The corrosion of the base metal may be markedly increased by the fastener material.
D. The plating on the fastener is rapidly consumed, leaving the bare fastener metal.
E. The corrosion of the fastener is increased by the base metal.

NOTE: Surface Treatment and environment can change activity. For a more thorough understanding of metal corrosion in construction materials, please consult a full listing of the galvanic series of metals and alloys.

Note: Reprinted from the Specialty Steel Industry of North America Stainless Steel Fasteners Designer's Handbook.

Table 2–7
Summary of Surface Preparation Specifications

SSPC Specification No.	Title	Description
SP1	Solvent Cleaning	Removal of oil, grease, dirt, soil, salts, and contaminants by cleaning with solvent, vapor, alkali, emulson, or steam.
SP2	Hand-Tool Cleaning	Removal of all loose rust, loose mill scale, and loose paint to degree specified, by hand-chipping, scraping, sanding, and wire brushing.
SP3	Power-Tool Cleaning	Removal of all loose rust, loose mill scale, and loose paint to degree specified, by power-tool chipping, descaling, sanding, wire brushing, and grinding.
SP5/NACE No.1	Metal Blast Cleaning	Removal of all visible rust, mill scale, paint, and foreign matter by blast-cleaning by wheel or nozzle (dry or wet) using sand, grit, or shot. (For very corrosive atmospheres where high cost of cleaning is warranted.)
SP6/NACE No.3	Commercial Blast-Cleaning	Blast-cleaning until at least two-thirds of the surface area is free of all visible residues. (For conditions where thoroughly cleaned surface is required.)
SP7/NACE No. 4	Brush-Off Blast-Cleaning	Blast-cleaning of all except tightly adhering residues of mill scale, rust, and coatings, exposing numerous evenly distributed flecks of underlying metal.
SP8	Pickling	Complete removal of rust and mill scale by acid-pickling, duplex-pickling, or electrolytic pickling.
SP10/NACE No.2	Near-White Blast-Cleaning	Blast-cleaning to nearly White Metal cleanliness, until at least 95% of the surface area is free of all visible residues. (For high humidity, chemical atmosphere, marine, or other corrosive environments.)
SP11	Power-Tool Cleaning to Bare Metal	Complete removal of all rust, scale, and paint by power tools, with resultant surface profile.

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of flexural members subject to uniaxial flexure without axial forces or torsion. For the design of members subject to biaxial flexure and/or flexure in combination with axial tension or compression and/or torsion, see Part 6. For flexural members that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

SECTION PROPERTIES AND AREAS

For Flexure

Flexural design properties are based upon the full cross section with no reduction for bolt holes when the limitations in AISC Specification Section F13.1(a) are satisfied. Otherwise, the flexural design properties are based upon a flexural rupture check given in AISC Specification Section F13.1(b).

For Shear

For shear, the area is determined per AISC Specification Chapter G.

FLEXURAL STRENGTH

The nominal flexural strength of W-shapes is illustrated as a function of the unbraced length, L_b, in Figure 3–1. The available strength is determined as ϕM_n or M_n/Ω, which must equal or exceed the required strength (bending moment), M_u or M_a, respectively. The available flexural strength, ϕM_n or M_n/Ω, is determined per AISC Specification Chapter F. User Note F1.1 outlines the sections of Chapter F and the corresponding limit states applicable to each member type.

Braced, Compact Flexural Members

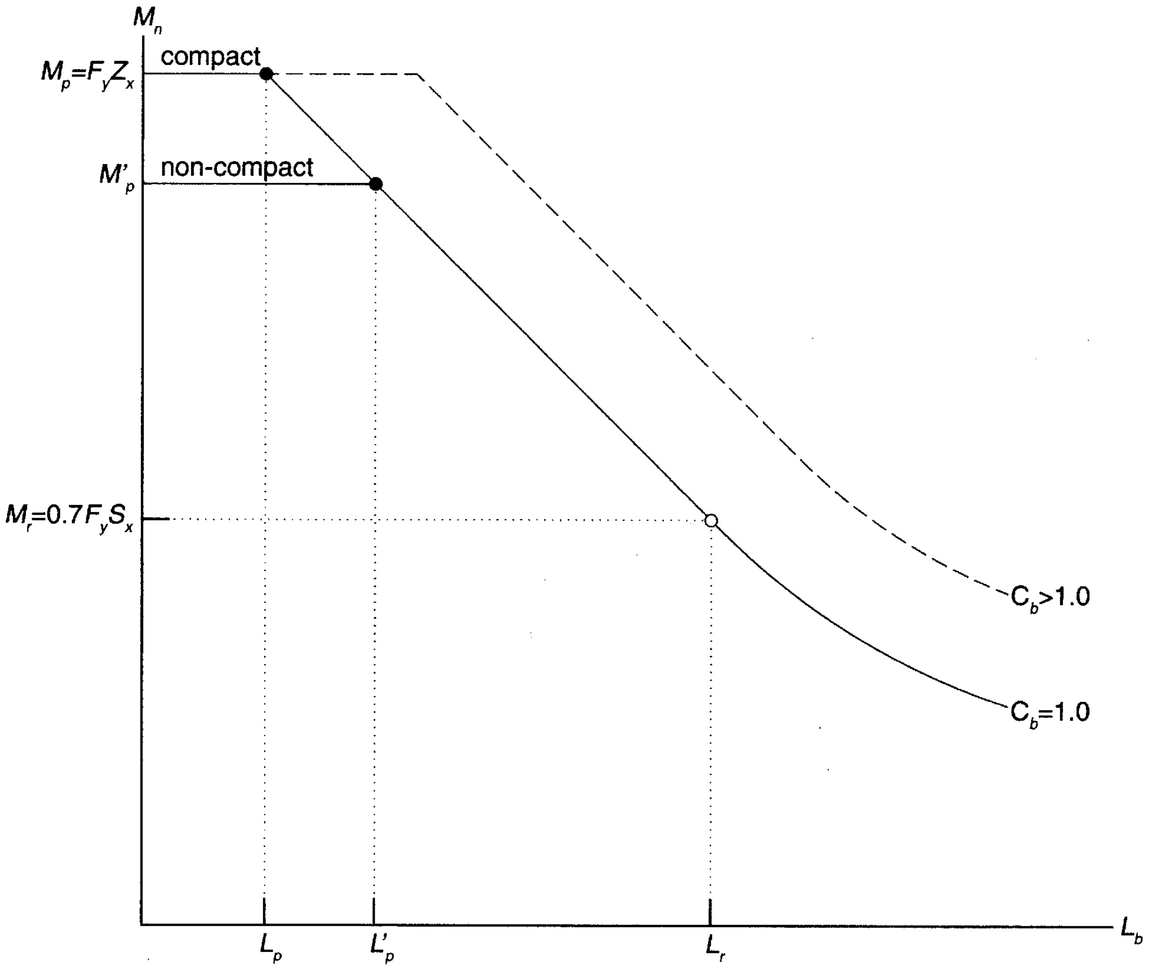
When flexural members are braced ($L_b \leq L_p$) and compact ($\lambda \leq \lambda_p$), yielding must be considered in the nominal moment strength of the member, in accordance with the requirements of AISC Specification Chapter F.

Unbraced Flexural Members

When flexural members are unbraced ($L_b > L_p$), have flange width-thickness ratios such that ($\lambda > \lambda_p$), or have web width-thickness ratios such that ($\lambda > \lambda_p$), lateral-torsional and elastic buckling effects must be considered in the calculation of the nominal moment strength of the member.

Non-Compact or Slender Cross-Sections

For flexural members that have width-thickness ratios such that ($\lambda>\lambda_p$), local buckling must be considered in the calculation of the nominal moment strength of the member.



$$L_p = 1.76r_y\sqrt{\frac{E}{F_y}}$$

$$L_r = 1.95r_{ts}\frac{E}{0.7F_y}\sqrt{\frac{Jc}{S_xh_o}}\sqrt{1+\sqrt{1+6.76\left(\frac{0.7F_y}{E}\frac{S_x}{J}\frac{h_o}{c}\right)^2}}$$

For non-compact cross-sections:

$$M'_p = M_p - \left(M_p - 0.7F_yS_x\right)\frac{\left(\lambda-\lambda_p\right)}{\left(\lambda_r-\lambda_p\right)}$$

$$L'_p = L_p + \left(L_r - L_p\right)\frac{\left(M_p - M'_p\right)}{\left(M_p - M_r\right)}$$

Figure 3–1. General available flexural strength of beams.

Available Flexural Strength for Weak-Axis Bending

The design of flexural members subject to weak-axis bending is similar to that for strong-axis bending, except that lateral-torsional buckling does not apply. See AISC Specification Section F6.

LOCAL BUCKLING

Determining the Width-Thickness Ratios of the Cross-Section

Flexural members are classified for flexure on the basis of the width-thickness ratios of the various elements of the cross-section. The width-thickness ratio λ is calculated for each element of the cross-section per AISC Specification Section B4.

Classification of Cross-Sections

Cross-sections are classified as follows:

- Flexural members are compact (the plastic moment can be reached without local buckling) when λ is equal to or less than λ_p and the flange(s) are continuously connected to the web(s).
- Flexural members are non-compact (local buckling will occur, but only after initial yielding) when λ exceeds λ_p but is equal to or less than λ_r.
- Flexural members are slender-element cross-sections (local buckling will occur prior to yielding) when λ exceeds λ_r.

The values of λ_p and λ_r are determined per AISC Specification Section B4.

LATERAL-TORSIONAL BUCKLING

Classification of Spans for Flexure

Flexural members bent about their strong axis are classified on the basis of the length L_b between braced points. Braced points are points at which support resistance against lateral-torsional buckling is provided per AISC Specification Appendix 6.3. Classifications are determined as follows:

- If $L_b \leq L_p$, flexural member is not subject to lateral-torsional buckling
- If $L_p < L_b \leq L_r$, flexural member is subject to inelastic lateral-torsional buckling
- If $L_b > L_r$, flexural member is subject to elastic lateral-torsional buckling

The values of L_p and L_r are determined per AISC Specification Chapter F. These values are presented in Tables 3–2, 3–6, 3–7, 3–8, and 3–9.

Lateral-torsional buckling does not apply to flexural members bent about their weak axis or HSS bent about either axis, per AISC Specification Sections F6, F7 and F8.

Consideration of Moment Gradient

When $L_b > L_p$, the moment gradient between braced points can be considered in the determination of the available strength using the beam bending coefficient C_b. In the case of a uniform moment between braced points causing single-curvature of the member, $C_b = 1$. This represents the worst case and C_b can be conservatively taken as unity for use with the maximum moment between braced points in all designs per AISC Specification Section F1. However, when desired, a non-uniform moment gradient between braced points can be considered using C_b calculated as given in AISC Specification Equation F1–1. Exceptions are provided as follows:

1. As an alternative, when the moment diagram between braced points is a straight line, C_b can be calculated as given in AISC Commentary Equation C–F1.1.
2. For cantilevered members where the free end is unbraced, C_b must be taken as unity per AISC Specification Section F1.
3. For tees with the stem in compression, C_b should be taken as unity as recommended in AISC Commentary Section F9.

AVAILABLE SHEAR STRENGTH

For flexural members, the available shear strength, ϕV_n or V_n/Ω, which must equal or exceed the required strength, V_u or V_a, respectively, is determined in accordance with the AISC Specification Chapter G.

STEEL W-SHAPE BEAMS WITH COMPOSITE SLABS

The following pertains to W-shapes with composite concrete slabs in regions of positive moment. For composite flexural members in regions of negative moment, see AISC Specification Chapter I. For further information on composite design and construction, see Viest et al. (1997).

Concrete Slab Effective Width

The effective width of a concrete slab acting compositely with a steel beam is determined per AISC Specification Section I3.1a.

Shear Stud Connectors

Material, placement and spacing requirements for shear stud connectors are given in AISC Specification Chapter I. The nominal shear strength, Q_n, of one shear stud connector is determined per AISC Specification Section I3.2d and is tabulated for common design conditions in Table 3–21.

Available Flexural Strength for Positive Moment

The available flexural strength of a composite beam subject to positive moment is determined per AISC Specification Section I3.2a assuming a uniform compressive stress of $0.85f_c'$ and zero tensile strength in the concrete, and a uniform stress of F_y in the tension area (and compression area, if any) of the steel section. The position of the plastic neutral axis (PNA) can then be determined by static equilibrium.

Per AISC Specification Section I3.2d, enough shear stud connectors must be provided between a point of maximum moment and the nearest point of zero moment to transfer the horizontal shear force V' between the steel beam and concrete slab, where V' is determined per AISC Specification Section I3.2d-1. For partial-composite design, the shear strength of the shear stud connectors ΣQ_n controls the available flexural strength of the composite flexural member.

Shored and Unshored Construction

The available flexural strength is identical for both shored and unshored construction. In unshored construction, issues such as lateral support during construction and construction-load deflection may require consideration.

Available Shear Strength

Per AISC Specification Section I3.1b, the available shear strength for composite beams is determined as illustrated previously for steel beams.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

The following other specification requirements and design considerations apply to the design of flexural members.

Special Requirements for Heavy Shapes and Plates

For beams with complete-joint-penetration groove welded joints and made from heavy shapes with a flange thickness exceeding 2 in., see AISC Specification Sections A3.1c

For built-up sections consisting of plates with a thickness exceeding 2-in., see Section A3.1d.

Serviceability

Serviceability requirements, per AISC Specification Chapter L, should be appropriate for the application. This includes an appropriate limit on the deflection of the flexural member and the vibration characteristics of the system of which the flexural member is a part. See also AISC Design Guide No. 3 *Serviceability Design Considerations for Low-Rise Buildings* (Fisher and West, 2004), AISC Design Guide No. 5 *Low- and Medium-Rise Steel Buildings* (Allison, 1991) and AISC Design Guide No. 11 *Floor Vibrations Due to Human Activity* (Murray, Allen and Ungar, 1997).

The maximum vertical deflection Δ, in., can be calculated using the equations given in Tables 3–22 and 3–23. Alternatively, for common cases of simple-span beams and I-shaped members and channels, the following equation can be used:

$$\Delta = ML^2 / (C_1 I_x)$$

where

M = maximum service-load moment, kip-ft
L = span length, ft
I_x = moment of inertia, in.^{4}
C_1 = loading constant (see Figure 3–2) which includes the numerical constants appropriate for the given loading pattern, E, which has units of ksi, and a ft-to-in. conversion factor of 1,728 in.^{3}/ft^3.

FLEXURAL DESIGN TABLES

Table 3–1. Beam Bending Coefficient C_b

Values of the beam bending coefficient C_b are given for various loading conditions on simple-span beams in Table 3–1.

W-SHAPE SELECTION TABLES

Table 3–2. W-Shapes—Selection by Z_x

W-shapes are sorted in descending order by strong-axis flexural strength and then grouped in ascending order by weight with the lightest W-shape in each range in bold. Strong-axis available strengths in flexure and shear are given for W-shapes with $F_y = 50$ ksi (ASTM A992). C_b is taken as unity.

For compact W-shapes, when $L_b \le L_p$, the strong-axis available flexural strength, M_{px}/Ω_b or $\phi_b M_{px}$, can be determined using the tabulated strength values. When $L_p < L_b \le L_r$, linearly interpolate between the available strength at L_p and the available strength at L_r as follows:

LRFD	ASD
$\phi_b M_n = C_b [\phi_b M_{px} - BF(L_b - L_p)] \le \phi_b M_{px}$	$\frac{M_n}{\Omega_b} = C_b \left[\frac{M_{px}}{\Omega_b} - BF(L_b - L_p)\right] \le \frac{M_{px}}{\Omega_b}$

When $L_b > L_r$, see Table 3–10. For non-compact W-shapes, the tabulated values of M_{px}/Ω_b, $\phi_b M_{px}$, and L_p have been adjusted to account for the non-compactness.

The strong-axis available shear strength, $\phi_v V_n$ or V_n/Ω_v can be determined using the tabulated value.

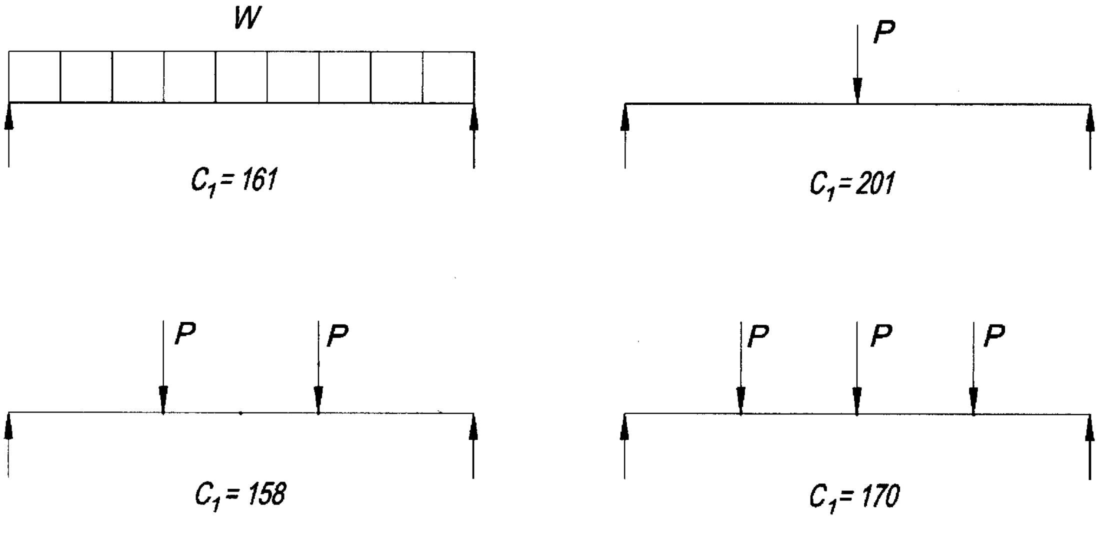


Figure 3–2. Loading constants for use in determining simple beam deflections.

Table 3–3. W-Shapes—Selection by I_x

W-shapes are sorted in descending order by strong-axis moment of inertia I_x and then grouped in ascending order by weight with the lightest W-shape in each range in bold.

Table 3–4. W-Shapes—Selection by Z_y

W-shapes are sorted in descending order by weak-axis flexural strength and then grouped in ascending order by weight with the lightest W-shape in each range in bold. Weak-axis available strengths in flexure are given for W-shapes with $F_y = 50$ ksi (ASTM A992). C_b is taken as unity.

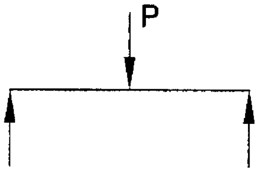
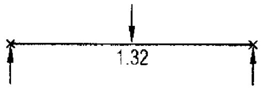
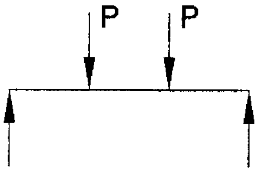
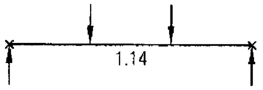
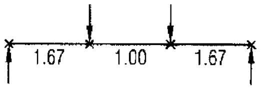
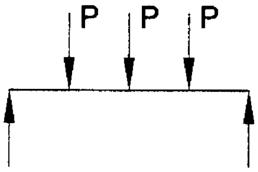
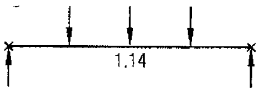
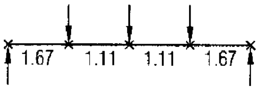
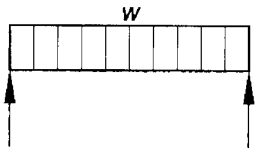
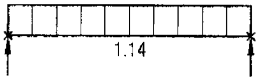
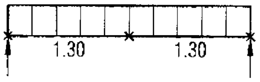
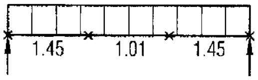
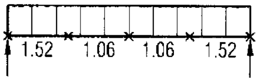
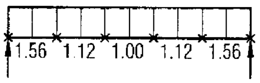
For non-compact W-shapes, the tabulated values of M_{py}/Ω_b and $\phi_b M_{py}$ have been adjusted to account for the non-compactness.

The weak-axis available shear strength must be checked independently.

Table 3–5. W-Shapes—Selection by I_y

W-shapes are sorted in descending order by weak-axis moment of inertia I_y and then grouped in ascending order by weight with the lightest W-shape in each range in bold.

Table 3–1
Values for C_b for Simply Supported Beams

Load	Lateral Bracing Along Span	C_b
	None Load at midpoint	
	At load point	
	None Loads at third points	
	At load points Loads symmetrically placed	
	None Loads at quarter points	
	At load points Loads at quarter points	
	None	
	At midpoint	
	At third points	
	At quarter points	
	At fifth points	

Note: Lateral bracing must always be provided at points of support per AISC Specification Chapter F.

F_y = 50 ksi

Table 3–2
W Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W36×800[h]	3650	9110	13700	5310	7980	47.5	71.4	14.9	94.8	64700	2030	3040
W36×652[h]	2910	7260	10900	4300	6460	46.8	70.4	14.5	77.8	50600	1620	2430
W40×593[h]	2760	6890	10400	4090	6140	55.5	83.5	13.4	63.8	50400	1540	2310
W36×529[h]	2330	5810	8740	3480	5220	46.5	70.0	14.1	64.4	39600	1280	1920
W40×503[h]	2310	5760	8660	3460	5200	54.7	82.2	13.1	55.3	41600	1290	1940
W36×487[h]	2130	5310	7990	3200	4800	46.1	69.3	14.0	60.0	36000	1180	1770
W40×431[h]	1960	4890	7350	2950	4440	53.6	80.6	12.9	49.0	34800	1110	1660
W36×441[h]	1910	4770	7160	2880	4330	45.2	68.0	13.8	55.5	32100	1060	1590
W27×539[h]	1890	4720	7090	2740	4120	26.1	39.2	12.9	88.6	25600	1280	1920
W40×397[h]	1800	4490	6750	2720	4100	52.3	78.7	12.9	46.6	32000	999	1500
W40×392[h]	1710	4270	6410	2510	3780	60.4	90.8	9.33	38.3	29900	1180	1760
W36×395[h]	1710	4270	6410	2600	3910	44.7	67.1	13.7	51.0	28500	937	1410
W40×372[h]	1680	4190	6300	2550	3830	51.6	77.6	12.7	44.5	29600	943	1410
W14×730[h]	1660	4140	6230	2240	3360	7.37	11.1	16.6	275	14300	1380	2060
W40×362[h]	1640	4090	6150	2480	3730	51.5	77.4	12.7	44.0	28900	908	1360
W44×335	1620	4040	6080	2460	3700	59.6	89.6	12.3	38.8	31100	902	1350
W33×387[h]	1560	3890	5850	2360	3540	38.4	57.7	13.3	53.3	24300	906	1360
W36×361[h]	1550	3870	5810	2360	3540	43.7	65.7	13.6	48.1	25700	851	1280
W14×665[h]	1480	3690	5550	2010	3020	7.12	10.7	16.3	253	12400	1220	1840
W40×324	1460	3640	5480	2240	3360	49.1	73.8	12.6	41.3	25600	803	1200
W30×391[h]	1450	3620	5440	2180	3280	31.3	47.1	13.0	58.8	20700	903	1350
W40×331[h]	1430	3570	5360	2110	3180	59.0	88.7	9.08	33.7	24700	995	1490
W33×354[h]	1420	3540	5330	2170	3260	37.5	56.4	13.2	49.9	22000	825	1240
W44×290	1410	3520	5290	2170	3260	54.9	82.5	12.3	37.0	27000	755	1130
W40×327[h]	1410	3520	5290	2100	3150	58.0	87.2	9.11	33.6	24500	963	1440
W36×330	1410	3520	5290	2170	3260	42.3	63.6	13.5	45.5	23300	768	1150
W40×297	1330	3320	4990	2040	3070	47.3	71.1	12.5	39.4	23200	741	1110
W30×357[h]	1320	3290	4950	1990	2990	31.2	47.0	12.9	54.5	18700	813	1220
W14×605[h]	1320	3290	4950	1820	2730	6.83	10.3	16.1	232	10800	1090	1630
W36×302	1280	3190	4800	1970	2970	40.5	60.9	13.5	43.6	21100	706	1060

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Z_X

Table 3–2 (continued)
W Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	*BF*		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W44×262	**1270**	**3170**	**4760**	**1940**	**2910**	**52.5**	**79.0**	**12.3**	**35.7**	**24100**	**680**	**1020**
W40×294	1270	3170	4760	1890	2840	57.0	85.7	9.01	31.5	21900	856	1280
W33×318	1270	3170	4760	1940	2910	36.9	55.4	13.1	46.5	19500	731	1100
W40×277	1250	3120	4690	1920	2890	45.8	68.9	12.6	38.8	21900	659	988
W27×368^h	1240	3090	4650	1850	2780	25.1	37.7	12.3	61.9	16200	839	1260
W40×278	1190	2970	4460	1780	2680	55.2	82.9	8.90	30.4	20500	823	1230
W36×282	1190	2970	4460	1830	2760	39.4	59.2	13.4	42.2	19600	657	985
W30×326^h	1190	2970	4460	1820	2730	30.3	45.6	12.7	50.7	16800	739	1110
W14×550^h	1180	2940	4430	1630	2440	6.67	10.0	15.9	213	9430	963	1450
W33×291	1160	2890	4350	1780	2680	36.0	54.1	13.0	43.9	17700	669	1000
W40×264	1130	2820	4240	1700	2550	54.1	81.4	8.90	29.7	19400	768	1150
W27×336^h	1130	2820	4240	1700	2550	25.1	37.7	12.2	56.9	14600	756	1130
W24×370^h	1130	2820	4240	1670	2510	19.9	29.9	11.6	69.2	13400	851	1280
W40×249	**1120**	**2790**	**4200**	**1730**	**2610**	**43.0**	**64.7**	**12.5**	**37.2**	**19600**	**591**	**886**
W44×230^v	**1100**	**2740**	**4130**	**1700**	**2550**	**47.1**	**70.9**	**12.1**	**34.4**	**20800**	**547**	**823**
W36×262	1100	2740	4130	1700	2550	38.4	57.7	13.3	40.6	17900	619	929
W30×292	1060	2640	3980	1620	2440	29.8	44.8	12.6	46.9	14900	653	980
W14×500^h	1050	2620	3940	1460	2200	6.42	9.65	15.6	196	8210	858	1290
W36×256	1040	2590	3900	1560	2350	46.5	70.0	9.36	31.5	16800	719	1080
W33×263	1040	2590	3900	1610	2410	34.6	51.9	12.9	41.6	15900	601	901
W36×247	1030	2570	3860	1590	2400	37.1	55.8	13.2	39.5	16700	587	880
W27×307^h	1030	2570	3860	1550	2330	25.2	37.8	12.0	52.6	13100	687	1030
W24×335^h	1020	2540	3830	1510	2270	20.1	30.2	11.4	63.0	11900	760	1140
W40×235	1010	2520	3790	1530	2300	51.0	76.7	8.97	28.4	17400	659	988
W40×215	**964**	**2410**	**3620**	**1500**	**2250**	**39.2**	**58.9**	**12.5**	**35.6**	**16700**	**507**	**760**
W36×231	963	2400	3610	1490	2240	35.8	53.7	13.1	38.6	15600	555	832
W30×261	943	2350	3540	1450	2180	29.2	43.9	12.5	43.4	13100	588	882
W33×241	940	2350	3530	1450	2180	33.2	49.8	12.8	39.7	14200	567	851
W36×232	936	2340	3510	1410	2120	44.6	67.1	9.25	29.9	15000	646	969
W27×281	936	2340	3510	1420	2140	24.6	36.9	12.0	49.2	11900	621	931
W14×455^h	936	2340	3510	1320	1980	6.20	9.31	15.5	179	7190	767	1150
W24×306^h	922	2300	3460	1380	2070	19.9	29.8	11.3	57.8	10700	684	1030
W40×211	**906**	**2260**	**3400**	**1370**	**2060**	**48.5**	**73.0**	**8.87**	**27.2**	**15500**	**591**	**886**

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

^{h} Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

^{v} Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

$F_y = 50$ ksi

Table 3–2 (continued)
W Shapes
Selection by Z_x

Z_X

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	*BF*		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W40×199	**869**	**2170**	**3260**	**1340**	**2020**	**37.2**	**55.9**	**12.2**	**34.3**	**14900**	**503**	**754**
W14×426^h	869	2170	3260	1230	1850	6.09	9.16	15.3	169	6600	700	1050
W33×221	857	2140	3210	1330	1990	31.8	47.8	12.7	38.2	12900	526	789
W27×258	852	2130	3200	1300	1960	24.2	36.4	11.9	45.9	10800	568	852
W30×235	847	2110	3180	1310	1960	28.3	42.5	12.4	40.9	11700	520	779
W24×279^h	835	2080	3130	1250	1880	19.7	29.6	11.2	53.4	9600	620	930
W36×210	833	2080	3120	1260	1890	42.5	63.8	9.11	28.5	13200	609	914
W14×398^h	801	2000	3000	1150	1720	5.96	8.96	15.2	158	6000	647	971
W40×183	**774**	**1930**	**2900**	**1180**	**1770**	**44.1**	**66.3**	**8.80**	**25.9**	**13200**	**507**	**760**
W33×201	773	1930	2900	1200	1800	30.2	45.3	12.6	36.8	11600	482	722
W27×235	772	1930	2900	1180	1780	23.9	35.9	11.8	42.9	9700	522	782
W36×194	767	1910	2880	1160	1740	40.6	61.0	9.04	27.6	12100	558	837
W18×311^h	754	1880	2830	1090	1640	11.2	16.8	10.4	81.2	6970	679	1020
W30×211	751	1870	2820	1160	1750	27.0	40.7	12.3	38.7	10300	480	719
W24×250	744	1860	2790	1120	1690	19.5	29.3	11.1	48.6	8490	548	822
W14×370^h	736	1840	2760	1060	1590	5.86	8.80	15.1	148	5440	593	890
W36×182	**718**	**1790**	**2690**	**1090**	**1640**	**39.1**	**58.8**	**9.01**	**27.0**	**11300**	**527**	**790**
W27×217	711	1770	2670	1100	1650	23.3	35.1	11.7	40.8	8910	472	708
W40×167	**693**	**1730**	**2600**	**1050**	**1580**	**41.8**	**62.9**	**8.48**	**24.8**	**11600**	**502**	**753**
W18×283^h	676	1690	2540	987	1480	11.0	16.6	10.3	73.8	6170	612	918
W30×191	675	1680	2530	1050	1580	25.8	38.7	12.2	36.9	9200	436	653
W24×229	675	1680	2530	1030	1540	19.2	28.9	11.0	45.2	7650	500	749
W14×342^h	672	1680	2520	975	1460	5.75	8.64	15.0	137	4900	540	810
W36×170	668	1670	2510	1010	1530	37.4	56.2	8.94	26.4	10500	492	738
W27×194	631	1570	2370	976	1470	22.5	33.8	11.6	38.2	7860	422	632
W33×169	629	1570	2360	959	1440	34.1	51.3	8.83	26.7	9290	453	680
W36×160	**624**	**1560**	**2340**	**947**	**1420**	**36.0**	**54.1**	**8.83**	**25.8**	**9760**	**468**	**702**
W18×258^h	611	1520	2290	898	1350	11.0	16.5	10.2	67.4	5510	549	824
W30×173	607	1510	2280	945	1420	24.4	36.6	12.1	35.5	8230	399	598
W24×207	606	1510	2270	927	1390	18.9	28.5	10.9	41.8	6820	447	671
W14×311^h	603	1500	2260	884	1330	5.63	8.46	14.8	125	4330	483	724
W12×336^h	603	1500	2260	844	1270	4.80	7.22	12.3	150	4060	597	896

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Table 3–2 (continued)
W Shapes
Selection by Z_x

$F_y = 50$ ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF	BF	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W40×149[v]	**598**	**1490**	**2240**	**896**	**1350**	**38.6**	**58.0**	**8.09**	**23.5**	**9800**	**432**	**650**
W36×150	581	1450	2180	880	1320	34.5	51.8	8.72	25.2	9040	448	672
W27×178	570	1420	2140	882	1330	21.7	32.7	11.5	36.3	7020	403	605
W33×152	559	1390	2100	851	1280	32.0	48.1	8.72	25.7	8160	425	638
W24×192	559	1390	2100	858	1290	18.7	28.0	10.8	39.6	6260	413	619
W18×234[h]	549	1370	2060	814	1220	10.8	16.2	10.1	61.5	4900	489	733
W14×283[h]	542	1350	2030	802	1200	5.53	8.31	14.7	114	3840	432	648
W12×305[h]	537	1340	2010	760	1140	4.66	7.00	12.1	137	3550	530	796
W21×201	530	1320	1990	805	1210	14.6	21.9	10.7	46.1	5310	419	629
W27×161	515	1280	1930	800	1200	20.8	31.3	11.4	34.7	6310	364	546
W33×141	**514**	**1280**	**1930**	**782**	**1180**	**30.4**	**45.8**	**8.58**	**25.0**	**7450**	**403**	**604**
W24×176	511	1270	1920	786	1180	18.3	27.6	10.7	37.4	5680	379	568
W36×135[v]	**509**	**1270**	**1910**	**767**	**1150**	**31.8**	**47.8**	**8.41**	**24.2**	**7800**	**383**	**576**
W30×148	500	1250	1880	761	1140	28.8	43.3	8.05	24.9	6680	399	598
W18×211	490	1220	1840	732	1100	10.7	16.1	9.96	55.8	4330	438	657
W14×257	487	1220	1830	725	1090	5.46	8.21	14.6	104	3400	385	577
W12×279[h]	481	1200	1800	686	1030	4.52	6.79	11.9	126	3110	485	728
W21×182	476	1190	1790	728	1090	14.3	21.6	10.6	42.6	4730	377	566
W24×162	468	1170	1760	723	1090	17.8	26.8	10.8	35.7	5170	353	529
W33×130	**467**	**1170**	**1750**	**709**	**1070**	**28.8**	**43.3**	**8.44**	**24.3**	**6710**	**384**	**576**
W27×146	464	1160	1740	723	1090	19.7	29.6	11.3	33.4	5660	331	497
W18×192	442	1100	1660	664	998	10.7	16.0	9.85	51.1	3870	391	586
W30×132	437	1090	1640	664	998	26.9	40.5	7.95	23.8	5770	373	559
W14×233	436	1090	1640	655	984	5.38	8.09	14.5	94.9	3010	343	515
W21×166	432	1080	1620	664	998	14.2	21.3	10.6	39.8	4280	337	506
W12×252[h]	428	1070	1610	617	927	4.40	6.62	11.8	114	2720	430	645
W24×146	418	1040	1570	648	974	17.1	25.8	10.6	33.7	4580	322	482
W33×118[v]	**415**	**1040**	**1560**	**627**	**942**	**26.7**	**40.2**	**8.19**	**23.5**	**5900**	**325**	**488**
W30×124	408	1020	1530	620	932	25.9	39.0	7.88	23.2	5360	353	529
W18×175	398	993	1490	601	903	10.6	15.9	9.75	46.7	3450	357	535
W27×129	395	986	1480	603	906	23.3	35.0	7.81	24.3	4760	337	506
W14×211	390	973	1460	590	887	5.31	7.99	14.4	86.4	2660	308	462
W12×230[h]	386	963	1450	561	843	4.32	6.49	11.7	105	2420	387	580

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

F_y = 50 ksi

Table 3–2 (continued)
W Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W30×116	**378**	**943**	**1420**	**575**	**864**	**24.7**	**37.2**	**7.74**	**22.6**	**4930**	**339**	**509**
W21×147	373	931	1400	575	864	13.8	20.7	10.4	36.3	3630	318	476
W24×131	370	923	1390	575	864	16.3	24.5	10.5	31.9	4020	296	444
W18×158	356	888	1340	541	814	10.5	15.7	9.68	42.8	3060	319	479
W14×193	355	886	1330	541	814	5.27	7.92	14.3	79.7	2400	276	413
W12×210	348	868	1310	510	767	4.24	6.38	11.6	96.0	2140	347	521
W30×108	**346**	**863**	**1300**	**522**	**785**	**23.7**	**35.6**	**7.59**	**22.0**	**4470**	**325**	**488**
W27×114	343	856	1290	522	785	21.7	32.6	7.70	23.1	4080	311	467
W21×132	333	831	1250	515	774	13.3	20.0	10.3	34.1	3220	284	426
W24×117	327	816	1230	508	764	15.3	23.1	10.4	30.4	3540	267	400
W18×143	322	803	1210	493	740	10.4	15.6	9.61	39.6	2750	285	427
W14×176	320	798	1200	491	738	5.22	7.84	14.2	73.2	2140	253	379
W30×99	**312**	**778**	**1170**	**470**	**706**	**22.2**	**33.3**	**7.42**	**21.4**	**3990**	**308**	**463**
W12×190	311	776	1170	459	690	4.18	6.28	11.5	87.3	1890	305	457
W21×122	307	766	1150	477	717	12.9	19.4	10.3	32.7	2960	260	390
W27×102	305	761	1140	466	701	20.2	30.3	7.59	22.2	3620	279	419
W18×130	290	724	1090	447	672	10.2	15.3	9.54	36.7	2460	258	387
W24×104	289	721	1080	451	677	14.3	21.5	10.3	29.2	3100	241	361
W14×159	287	716	1080	444	667	5.18	7.79	14.1	66.7	1900	223	335
W30×90^v	**283**	**706**	**1060**	**428**	**643**	**20.5**	**30.9**	**7.38**	**20.9**	**3610**	**249**	**375**
W24×103	280	699	1050	428	643	18.2	27.4	7.03	21.9	3000	270	405
W21×111	279	696	1050	435	654	12.4	18.7	10.2	31.3	2670	237	355
W27×94	278	694	1040	424	638	19.1	28.8	7.49	21.6	3270	264	396
W12×170	275	686	1030	410	617	4.11	6.18	11.4	78.5	1650	269	404
W18×119	262	654	983	403	606	10.1	15.2	9.50	34.3	2190	249	373
W14×145	260	649	975	405	609	5.11	7.68	14.1	61.7	1710	201	302
W24×94	254	634	953	388	583	17.3	26.0	6.99	21.2	2700	250	376
W21×101	253	631	949	396	596	11.8	17.7	10.2	30.1	2420	214	320
W27×84	**244**	**609**	**915**	**372**	**559**	**17.6**	**26.4**	**7.31**	**20.8**	**2850**	**246**	**369**
W12×152	243	606	911	365	549	4.07	6.11	11.3	70.6	1430	239	358
W14×132	234	584	878	365	549	5.13	7.70	13.3	56.0	1530	189	284
W18×106	230	574	863	356	536	9.70	14.6	9.40	31.8	1910	221	332

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

Table 3–2 (continued)
W Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	*BF*	*BF*	L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W24×84	**224**	**559**	**840**	**342**	**515**	**16.2**	**24.3**	**6.89**	**20.3**	**2370**	**227**	**340**
W21×93	221	551	829	335	504	14.6	21.9	6.50	21.3	2070	251	376
W12×136	214	534	803	325	488	4.01	6.03	11.2	63.3	1240	212	318
W14×120	212	529	795	332	499	5.09	7.64	13.2	52.0	1380	171	256
W18×97	211	526	791	328	494	9.45	14.2	9.36	30.3	1750	199	298
W24×76	**200**	**499**	**750**	**307**	**462**	**15.0**	**22.5**	**6.78**	**19.6**	**2100**	**210**	**316**
W16×100	198	494	743	306	459	7.90	11.9	8.87	32.7	1490	199	298
W21×83	196	489	735	299	449	13.8	20.8	6.46	20.2	1830	221	331
W14×109	192	479	720	302	454	5.02	7.54	13.2	48.4	1240	150	226
W18×86	186	464	698	290	436	9.04	13.6	9.29	28.5	1070	177	265
W12×120	186	464	698	285	428	3.95	5.93	11.1	56.5	1530	186	279
W24×68	**177**	**442**	**664**	**269**	**404**	**14.1**	**21.2**	**6.61**	**18.8**	**1830**	**197**	**295**
W16×89	175	437	656	271	407	7.74	11.6	8.80	30.2	1300	176	264
W14×99^f	173	430	646	274	412	4.89	7.35	13.5	45.3	1110	137	206
W21×73	172	429	645	264	396	12.9	19.4	6.39	19.2	1600	193	290
W12×106	164	409	615	253	381	3.93	5.90	11.0	50.7	933	157	236
W18×76	163	407	611	255	383	8.49	12.8	9.22	27.1	1330	155	232
W21×68	**160**	**399**	**600**	**245**	**368**	**12.5**	**18.8**	**6.36**	**18.7**	**1480**	**182**	**273**
W14×90^f	157	382	573	250	375	4.80	7.22	15.2	42.6	999	123	185
W24x62	**153**	**382**	**574**	**229**	**344**	**16.0**	**24.1**	**4.87**	**14.4**	**1550**	**204**	**306**
W16×77	150	374	563	234	352	7.34	11.0	8.72	27.8	1110	150	225
W12×96	147	367	551	229	344	3.87	5.81	10.9	46.6	833	140	210
W10×112	147	367	551	220	331	2.68	4.02	9.47	64.3	716	172	257
W18×71	146	364	548	222	333	10.5	15.7	6.00	19.6	1170	183	274
W21×62	**144**	**359**	**540**	**222**	**333**	**11.6**	**17.4**	**6.25**	**18.1**	**1330**	**168**	**252**
W14×82	139	347	521	215	323	5.43	8.16	8.76	33.1	881	146	219
W24×55^v	**134**	**334**	**503**	**199**	**299**	**14.8**	**22.2**	**4.73**	**13.9**	**1350**	**167**	**251**
W18×65	133	332	499	204	307	9.92	14.9	5.97	18.8	1070	165	248
W12×87	132	329	495	206	310	3.84	5.76	10.8	43.0	740	129	194
W16×67	130	324	488	204	307	6.91	10.4	8.69	26.1	954	129	194
W10×100	130	324	488	196	294	2.66	4.01	9.36	57.7	623	151	226
W21×57	129	322	484	194	291	13.4	20.1	4.77	14.3	1170	171	256

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

F_y = 50 ksi

Table 3–2 (continued)
W Shapes
Selection by Z_x

Z_x

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	BF		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W21×55	126	314	473	192	289	10.8	16.3	6.11	17.4	1140	156	234
W14×74	126	314	473	196	294	5.34	8.03	8.76	31.0	795	128	191
W18×60	123	307	461	189	284	9.64	14.5	5.93	18.2	984	151	227
W12×79	119	297	446	187	281	3.77	5.67	10.8	39.9	662	116	175
W14×68	115	287	431	180	270	5.20	7.81	8.69	29.3	722	117	175
W10×88	113	282	424	172	259	2.63	3.95	9.29	51.1	534	131	197
W18×55	112	279	420	172	258	9.26	13.9	5.90	17.5	890	141	212
W21×50	110	274	413	165	248	12.2	18.3	4.59	13.6	984	158	237
W12×72	108	269	405	170	256	3.72	5.59	10.7	37.4	597	105	158
W21×48^f	107	265	398	162	244	9.78	14.7	6.09	16.6	959	144	217
W16×57	105	262	394	161	242	7.98	12.0	5.65	18.3	758	141	212
W14×61	102	254	383	161	242	4.96	7.46	8.65	27.5	640	104	156
W18×50	101	252	379	155	233	8.69	13.1	5.83	17.0	800	128	192
W10×77	97.6	244	366	150	225	2.59	3.90	9.18	45.2	455	112	169
W12×65^f	96.8	237	356	154	231	3.60	5.41	11.9	35.1	533	94.5	142
W21×44	95.4	238	358	143	214	11.2	16.8	4.45	13.0	843	145	217
W16×50	92.0	230	345	141	213	7.59	11.4	5.62	17.2	659	124	185
W18×46	90.7	226	340	138	207	9.71	14.6	4.56	13.7	712	130	195
W14×53	87.1	217	327	136	204	5.27	7.93	6.78	22.2	541	103	155
W12×58	86.4	216	324	136	205	3.76	5.66	8.87	29.9	475	87.8	132
W10×68	85.3	213	320	132	199	2.57	3.86	9.15	40.6	394	97.8	147
W16×45	82.3	205	309	127	191	7.16	10.8	5.55	16.5	586	111	167
W18×40	78.4	196	294	119	180	8.86	13.3	4.49	13.1	612	113	169
W14×48	78.4	196	294	123	184	5.10	7.66	6.75	21.1	484	93.8	141
W12×53	77.9	194	292	123	185	3.65	5.48	8.76	28.2	425	83.2	125
W10×60	74.6	186	280	116	175	2.53	3.80	9.08	36.6	341	85.8	129
W16×40	73.0	182	274	113	170	6.69	10.1	5.55	15.9	518	97.7	146
W12×50	71.9	179	270	112	169	3.97	5.97	6.92	23.9	391	90.2	135
W8×67	70.1	175	263	105	159	1.73	2.60	7.49	47.7	272	103	154
W14×43	69.6	174	261	109	164	4.82	7.24	6.68	20.0	428	83.3	125
W10×54	66.6	166	250	105	158	2.49	3.74	9.04	33.7	303	74.7	112

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.

Z_X

Table 3–2 (continued)
W Shapes
Selection by Z_x

F_y = 50 ksi

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	*BF*		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W18×35	66.5	166	249	101	151	8.07	12.1	4.31	12.4	510	106	159
W12×45	64.2	160	241	101	151	3.83	5.75	6.89	22.4	348	80.8	121
W16×36	64.0	160	240	98.7	148	6.19	9.31	5.37	15.2	448	93.6	140
W14×38	61.5	153	231	95.4	143	5.39	8.10	5.47	16.2	385	87.4	131
W10×49	60.4	151	227	95.4	143	2.44	3.67	8.97	31.6	272	68.0	102
W8×58	59.8	149	224	90.8	137	1.70	2.56	7.42	41.7	228	89.3	134
W12×40	57.0	142	214	89.9	135	3.66	5.50	6.85	21.1	307	70.4	106
W10×45	54.9	137	206	85.8	129	2.59	3.89	7.10	26.9	248	70.7	106
W14×34	54.6	136	205	84.9	128	5.05	7.59	5.40	15.6	340	79.7	120
W16×31	54.0	135	203	82.4	124	6.76	10.2	4.13	11.9	375	87.3	131
W12×35	51.2	128	192	79.6	120	4.28	6.43	5.44	16.7	285	75.0	113
W8×48	49.0	122	184	75.4	113	1.68	2.53	7.35	35.2	184	68.0	102
W14×30	47.3	118	177	73.4	110	4.65	6.99	5.26	14.9	291	74.7	112
W10×39	46.8	117	176	73.5	111	2.51	3.77	6.99	24.2	209	62.5	93.7
W16×26^v	44.2	110	166	67.1	101	5.96	8.96	3.96	11.2	301	70.5	106
W12×30	43.1	108	162	67.4	101	3.92	5.89	5.37	15.6	238	64.2	96.3
W14×26	40.2	100	151	61.7	92.7	5.32	7.99	3.81	11.1	245	70.9	106
W8×40	39.8	99.3	149	62.0	93.2	1.64	2.47	7.21	29.9	146	59.4	89.1
W10×33	38.8	96.8	146	61.1	91.9	2.39	3.59	6.85	21.8	171	56.4	84.7
W12×26	37.2	92.8	140	58.3	87.7	3.61	5.42	5.33	14.9	204	56.2	84.3
W10×30	36.6	91.3	137	56.6	85.0	3.08	4.62	4.84	16.1	170	62.8	94.2
W8×35	34.7	86.6	130	54.5	81.9	1.62	2.43	7.17	27.0	127	50.3	75.5
W14×22	33.2	82.8	125	50.6	76.1	4.75	7.14	3.67	10.4	199	63.2	94.8
W10×26	31.3	78.1	117	48.7	73.2	2.90	4.36	4.80	14.9	144	53.7	80.6
W8×31^f	30.4	75.8	114	48.0	72.2	1.58	2.37	7.18	24.8	110	45.6	68.4
W12×22	29.3	73.1	110	44.4	66.7	4.65	6.99	3.00	9.17	156	64.0	96.0
W8×28	27.2	67.9	102	42.4	63.8	1.66	2.50	5.72	21.0	98.0	45.9	68.9
W10×22	26.0	64.9	97.5	40.5	60.9	2.68	4.02	4.70	13.8	118	48.8	73.2
W12×19	24.7	61.6	92.6	37.2	55.9	4.27	6.43	2.90	8.62	130	57.2	85.7
W8x24	23.1	57.6	86.6	36.5	54.9	1.59	2.39	5.69	19.0	82.7	38.9	58.3
W10×19	21.6	53.9	81.0	32.8	49.3	3.17	4.77	3.09	9.72	96.3	51.2	76.8
W8×21	20.4	50.9	76.5	31.8	47.8	1.86	2.79	4.45	14.8	75.3	41.4	62.1

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

F_y = 50 ksi

Table 3–2 (continued)
W Shapes
Selection by Z_x

Z_X

Shape	Z_x	M_{px}/Ω_b	$\phi_b M_{px}$	M_{rx}/Ω_b	$\phi_b M_{rx}$	*BF*		L_p	L_r	I_x	V_{nx}/Ω_v	$\phi_v V_{nx}$
		kip-ft	kip-ft	kip-ft	kip-ft	kips	kips				kips	kips
	in.^{3}	ASD	LRFD	ASD	LRFD	ASD	LRFD	ft	ft	in.^{4}	ASD	LRFD
W12×16	**20.1**	**50.1**	**75.4**	**29.9**	**44.9**	**3.82**	**5.75**	**2.73**	**8.03**	**103**	**52.8**	**79.1**
W10×17	18.7	46.7	70.1	28.3	42.5	2.99	4.49	2.98	9.13	81.9	48.5	72.8
W12×14^v	**17.4**	**43.4**	**65.2**	**26.0**	**39.1**	**3.42**	**5.15**	**2.66**	**7.74**	**88.6**	**42.8**	**64.3**
W8×18	17.0	42.4	63.8	26.5	39.9	1.74	2.61	4.34	13.50	61.9	37.4	56.2
W10×15	16.0	39.9	60.0	24.1	36.2	2.75	4.14	2.86	8.61	68.9	46.0	69.0
W8×15	13.6	33.9	51.0	20.6	31.0	1.92	2.88	3.09	10.00	48.0	39.7	59.6
W10×12^f	**12.6**	**31.2**	**46.9**	**19.0**	**28.6**	**2.35**	**3.53**	**2.87**	**8.05**	**53.8**	**37.5**	**56.3**
W8×13	11.4	28.4	42.8	17.3	26.0	1.76	2.65	2.98	9.30	39.6	36.8	55.1
W8×10^f	**8.9**	**21.9**	**32.9**	**13.6**	**20.5**	**1.52**	**2.28**	**3.14**	**8.56**	**30.8**	**26.8**	**40.2**

ASD	LRFD
Ω_b = 1.67 Ω_v = 1.50	ϕ_b = 0.90 ϕ_v = 1.00

[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi, Ω_v = 1.67, ϕ_v = 0.90.

I_x

Table 3–3
W Shapes
Selection by I_x

Shape	I_x	Shape	I_x	Shape	I_x	Shape	I_x
	in.^{4}		in.^{4}		in.^{4}		in.^{4}
W36×800[h]	**64700**	**W44×230**	**20800**	**W40×167**	**11600**	**W33×118**	**5900**
		W30×391[h]	20700	W33×201	11600	W30×132	5770
W36×652[h]	**50600**	W40×278	20500	W36×182	11300	W24×176	5680
		W40×249	19600	W27×258	10800	W27×146	5660
W40×593[h]	**50400**	W36×282	19600	W14×605[h]	10800	W18×258[h]	5510
		W33×318	19500	W24×306[h]	10700	W14×370[h]	5440
W40×503[h]	**41600**	W40×264	19400	W36×170	10500	W30×124	5360
W36×529[h]	39600	W30×357[h]	18700	W30×211	10300	W21×201	5310
		W36×262	17900			W24×162	5170
W36×487[h]	**36000**	W33×291	17700	**W40×149**	**9800**		
		W40×235	17400	W36×160	9760	**W30×116**	**4930**
W40×431[h]	**34800**	W36×256	16800	W27×235	9700	W18×234[h]	4900
W36×441[h]	32100	W30×326[h]	16800	W24×279[h]	9600	W14×342[h]	4900
				W14×550[h]	9430	W27×129	4760
W40×397[h]	**32000**	**W40×215**	**16700**	W33×169	9290	W21×182	4730
		W36×247	16700	W30×191	9200	W24×146	4580
W44×335	**31100**	W27×368[h]	16200	W36×150	9040		
W40×392[h]	29900	W33×263	15900	W27×217	8910	**W30×108**	**4470**
W40×372[h]	29600	W36×231	15600	W24×250	8490	W18×211	4330
W40×362[h]	28900			W30×173	8230	W14×311[h]	4330
W36×395[h]	28500	**W40×211**	**15500**	W14×500[h]	8210	W21×166	4280
		W36×232	15000	W33×152	8160	W27×114	4080
W44×290	**27000**			W27×194	7860	W12×336[h]	4060
W36×361[h]	25700	**W40×199**	**14900**			W24×131	4020
W40×324	25600	W30×292	14900	**W36×135**	**7800**		
W27×539[h]	25600	W27×336[h]	14600	W24×229	7650	**W30×99**	**3990**
W40×331[h]	24700	W14×730[h]	14300	W33×141	7450	W18×192	3870
W40×327[h]	24500	W33×241	14200	W14×455[h]	7190	W14×283[h]	3840
W33×387[h]	24300	W24×370[h]	13400	W27×178	7020	W21×147	3630
				W18×311[h]	6970	W27×102	3620
W44×262	**24100**	**W40×183**	**13200**	W24×207	6820		
W36×330	23300	W36×210	13200				
W40×297	23200	W30×261	13100	**W33×130**	**6710**		
W33×354[h]	22000	W27×307[h]	13100	W30×148	6680		
W40×277	21900	W33×221	12900	W14×426[h]	6600		
W40×294	21900	W14×665[h]	12400	W27×161	6310		
W36×302	21100	W36×194	12100	W24×192	6260		
		W27×281	11900	W18×283[h]	6170		
		W24×335[h]	11900	W14×398[h]	6000		
		W30×235	11700				

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Table 3–3 (continued)
W Shapes
Selection by I_x

I_X

Shape	I_x in.^{4}	Shape	I_x in.^{4}	Shape	I_x in.^{4}	Shape	I_x in.^{4}
W30×90	**3610**	**W24×68**	**1830**	**W21×44**	**843**	**W16×26**	**301**
W12×305[h]	3550	W21×83	1830	W12×96	833	W14×30	291
W24×117	3540	W18×97	1750	W18×50	800	W12×35	285
W18×175	3450	W14×145	1710	W14×74	795	W10×49	272
W14×257	3400	W12×170	1650	W16×57	758	W8×67	272
W27×94	3270	W21×73	1600	W12×87	740	W10×45	248
W21×132	3220			W14×68	722		
W12×279[h]	3110	**W24×62**	**1550**	W10×112	716	**W14×26**	**245**
W24×104	3100	W18×86	1530	W18×46	712	W12×30	238
W18×158	3060	W14×132	1530	W12×79	662	W8×58	228
W14×233	3010	W16×100	1490	W16×50	659	W10×39	209
W24×103	3000	W21×68	1480	W14×61	640		
W21×122	2960	W12×152	1430	W10×100	623	**W12×26**	**204**
		W14×120	1380				
W27×84	**2850**			**W18×40**	**612**	**W14×22**	**199**
W18×143	2750	**W24×55**	**1350**	W12×72	597	W8×48	184
W12×252[h]	2720	W21×62	1330	W16×45	586	W10×33	171
W24×94	2700	W18×76	1330	W14×53	541	W10×30	170
W21×111	2670	W16×89	1300	W10×88	534		
W14×211	2660	W14×109	1240	W12×65	533	**W12×22**	**156**
W18×130	2460	W12×136	1240			W8×40	146
W21×101	2420	W21×57	1170	**W16×40**	**518**	W10×26	144
W12×230[h]	2420	W18×71	1170				
W14×193	2400			**W18×35**	**510**	**W12×19**	**130**
		W21×55	**1140**	W14×48	484	W8×35	127
W24×84	**2370**	W16×77	1110	W12×58	475	W10×22	118
W18×119	2190	W14×99	1110	W10×77	455	W8×31	110
W14×176	2140	W18×65	1070	W16×36	448		
W12×210	2140	W12×120	1070	W14×43	428	**W12×16**	**103**
		W14×90	999	W12×53	425	W8×28	98.0
W24×76	**2100**			W10×68	394	W10×19	96.3
W21×93	2070	**W21×50**	**984**	W12×50	391		
W18×106	1910	W18×60	984	W14×38	385	**W12×14**	**88.6**
W14×159	1900					W8×24	82.7
W12×190	1890	**W21×48**	**959**	**W16×31**	**375**	W10×17	81.9
		W16×67	954	W12×45	348	W8×21	75.3
		W12×106	933	W10×60	341	W10×15	68.9
		W18×55	890	W14×34	340	W8×18	61.9
		W14×82	881	W12×40	307		
				W10×54	303	**W10×12**	**53.8**
						W8×15	48.0
						W8×13	39.6
						W8×10	**30.8**

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Z_y

Table 3–4
W Shapes
Selection by Z_y

F_y = 50 ksi

Shape	Z_y	M_{py}/Ω_b	$\phi_b M_{py}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W14×730^h	**816**	**2040**	**3060**
W36×800	743	1850	2790
W14×665^h	**730**	**1820**	**2740**
W14×605^h	**652**	**1630**	**2450**
W14×550^h	**583**	**1450**	**2190**
W36×652^h	581	1450	2180
W14×500^h	**522**	**1300**	**1960**
W40×593^h	481	1200	1800
W14×455^h	**468**	**1170**	**1760**
W36×529^h	454	1130	1700
W27×539^h	437	1090	1640
W14×426^h	**434**	**1080**	**1630**
W36×487^h	412	1030	1550
W14×398^h	**402**	**1000**	**1510**
W40×503^h	394	983	1480
W14×370^h	**370**	**923**	**1390**
W36×441^h	368	918	1380
W14×342^h	**338**	**843**	**1270**
W40×431^h	328	818	1230
W36×395^h	325	811	1220
W33×387^h	312	778	1170
W30×391^h	310	773	1160
W14×311^h	**304**	**758**	**1140**
W40×397^h	300	749	1130
W36×361^h	293	731	1100
W33×354^h	282	704	1060
W30×357^h	279	696	1050
W27×368^h	279	696	1050
W40×372^h	277	691	1040
W14×283^h	**274**	**684**	**1030**
W12×336^h	274	684	1030
W40×362^h	270	674	1010
W24×370^h	267	666	1000
W36×330	265	661	994
W30×326^h	252	629	945
W27×336^h	252	629	945
W33×318	250	624	938
W14×257	**246**	**614**	**923**
W12×305^h	244	609	915
W36×302	241	601	904
W40×324	239	596	896
W24×335^h	238	594	893
W44×335	236	589	885
W27×307^h	227	566	851
W33×291	226	564	848
W36×282	223	556	836
W30×292	223	556	836
W14×233	**221**	**551**	**829**
W12×279^h	220	549	825
W40×297	215	536	806
W24×306^h	214	534	803
W40×392^h	212	519	780
W18×311^h	207	516	776
W27×281	206	514	773
W44×290	205	511	769
W40×277	204	509	765
W36×262	204	509	765
W33×263	202	504	758
W14×211	**198**	**494**	**743**
W30×261	196	489	735
W12×252^h	196	489	735
W24×279^h	193	482	724
W36×247	190	474	713
W27×258	187	467	701
W18×283^h	185	462	694
W44×262	182	454	683
W40×249	182	454	683
W33×241	182	454	683
W14×193	**180**	**449**	**675**
W12×230^h	177	442	664
W36×231	176	439	660
W30×235	175	437	656
W40×331^h	172	423	636
W24×250	171	427	641
W27×235	168	419	630
W18×258^h	166	414	623
W33×221	164	409	615
W14×176	**163**	**407**	**611**
W12×210	159	397	596
W44×230^f	157	392	589
W40×215	156	389	585
W30×211	155	387	581
W27×217	154	384	578
W24×229	154	384	578
W40×294	150	373	561
W18×234^h	149	372	559
W33×201	147	367	551

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

F_y = 50 ksi

Table 3–4 (continued)
W Shapes
Selection by Z_y

Z_y

Shape	Z_y	M_{py}/Ω_b	$\phi_b M_{py}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W14×159	**146**	**364**	**548**
W12×190	143	357	536
W40×278	140	348	523
W30×191	138	344	518
W40×199	137	342	514
W36×256	137	342	514
W24×207	137	342	514
W27×194	136	339	510
W21×201	133	332	499
W14×145	**133**	**332**	**499**
W40×264	132	329	4905
W18×211	132	329	495
W24×192	126	314	473
W12×170	126	314	473
W30×173	123	307	461
W36×232	122	304	458
W27×178	122	304	458
W21×182	119	297	446
W18×192	119	297	446
W40×235	118	294	443
W24×176	115	287	431
W14×132	**113**	**282**	**424**
W12×152	111	277	416
W27×161	109	272	409
W21×166	108	269	405
W36×210	107	267	401
W18×175	106	264	398
W40×211	105	262	394
W24×162	105	262	394
W14×120	**102**	**254**	**383**
W12×136	98.0	245	368
W36×194	97.7	244	366
W27×146	97.7	244	366
W18×158	94.8	237	356
W24×146	93.2	233	350

Shape	Z_y	M_{py}/Ω_b	$\phi_b M_{py}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W14×109	**92.7**	**231**	**348**
W21×147	92.6	231	347
W36×182	90.7	226	340
W40×183	88.3	220	331
W18×143	85.4	213	320
W12×120	85.4	213	320
W33×169	84.4	211	317
W36×170	83.8	209	314
W14×99[f]	**83.6**	**207**	**311**
W21×132	82.3	205	309
W24×131	81.5	203	306
W36×160	77.3	193	290
W18×130	76.7	191	288
W40×167	76.0	190	285
W21×122	75.6	189	283
W14×90[f]	**75.6**	**181**	**273**
W12×106	75.1	187	282
W33×152	73.9	184	277
W24×117	71.4	178	268
W36×150	70.9	177	266
W10×112	69.2	173	260
W18×119	69.1	172	259
W21×111	68.2	170	256
W30×148	68.0	170	255
W12×96	67.5	168	253
W33×141	66.9	167	251
W24×104	62.4	156	234
W40×149	62.2	155	233
W21×101	61.7	154	231
W10×100	61.0	152	229
W18×106	60.5	151	227

Shape	Z_y	M_{py}/Ω_b	$\phi_b M_{py}$
		kip-ft	kip-ft
	in.^{3}	ASD	LRFD
W12×87	**60.4**	**151**	**227**
W36×135	59.7	149	224
W33×130	59.5	148	223
W30×132	58.4	146	219
W27×129	57.6	144	216
W18×97	55.3	138	207
W16×100	54.9	137	206
W12×79	**54.3**	**135**	**204**
W30×124	54.0	135	203
W10×88	53.1	132	199
W33×118	51.3	128	192
W27×114	49.3	123	185
W30×116	49.2	123	185
W12×72	**49.2**	**123**	**185**
W18×86	48.4	121	182
W16×89	48.1	120	180
W10×77	45.9	115	172
W14×82	44.8	112	168
W12×65[f]	**44.1**	**107**	**161**
W30×108	43.9	110	165
W27×102	43.4	108	163
W18×76	42.2	105	158
W24×103	41.5	104	156
W16×77	41.1	103	154
W14×74	40.5	101	152
W10×68	40.1	100	150
W27×94	38.8	96.8	146
W30×99	38.6	96.3	145
W24×94	37.5	93.6	141
W14×68	36.9	92.1	138
W16×67	35.5	88.6	133

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

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[f] Shape exceeds compact limit for flexure with F_y = 50 ksi.

Z_y

Table 3-4 (continued)
W Shapes
Selection by Z_y

$F_y = 50$ ksi

Shape	Z_y in.^{3}	M_{py}/Ω_b kip-ft ASD	$\phi_b M_{py}$ kip-ft LRFD
W10×60	**35.0**	**87.3**	**131**
W30×90	34.7	**86.6**	130
W21×93	34.7	**86.6**	130
W27×84	33.2	**82.8**	125
W14×61	32.8	**81.8**	123
W8×67	32.7	**81.6**	123
W24×84	32.6	**81.3**	122
W12×58	**32.5**	**81.1**	**122**
W10×54	**31.3**	**78.1**	**117**
W21×83	30.5	**76.1**	114
W12×53	**29.1**	**72.6**	**109**
W24×76	28.6	**71.4**	107
W10×49	**28.3**	**70.6**	**106**
W8×58	27.9	**69.6**	105
W21×73	26.6	**66.4**	99.8
W18×71	24.7	**61.6**	92.6
W24×68	24.5	**61.1**	91.9
W21×68	24.4	**60.9**	91.5
W8×48	**22.9**	**57.1**	**85.9**
W18×65	22.5	**56.1**	84.4
W14×53	22.0	**54.9**	82.5
W21×62	21.7	**54.1**	81.4
W12×50	21.3	**53.1**	79.9
W18×60	20.6	**51.4**	77.3
W10×45	**20.3**	**50.6**	**76.1**
W14×48	19.6	**48.9**	73.5
W12×45	**19.0**	**47.4**	**71.3**
W16×57	18.9	**47.2**	70.9
W18×55	18.5	**46.2**	69.4
W8×40	**18.5**	**46.2**	**69.4**
W21×55	18.4	**45.9**	69.0
W14×43	17.3	**43.2**	64.9
W10×39	**17.2**	**42.9**	**64.5**
W12×40	16.8	**41.9**	63.0
W18×50	16.6	**41.4**	62.3
W16×50	16.3	**40.7**	61.1
W8×35	**16.1**	**40.2**	**60.4**
W24×62	15.7	**39.1**	58.8
W21×48[f]	14.9	**36.7**	55.2
W21×57	14.8	**36.9**	55.5
W16×45	14.5	**36.2**	54.4
W8×31[f]	**14.1**	**35.1**	**52.8**
W10×33	14.0	**34.9**	52.5
W24×55	13.3	**33.1**	49.8
W16×40	12.7	**31.7**	47.6
W21×50	12.2	**30.4**	45.8
W14×38	12.1	**30.2**	45.4
W18×46	11.7	**29.2**	43.9
W12×35	11.5	**28.7**	43.1
W16×36	10.8	**26.9**	40.5
W14×34	10.6	**26.4**	39.8
W21×44	10.2	**25.4**	38.2
W8×28	**10.1**	**25.2**	**37.9**
W18×40	10.0	**25.0**	37.5
W12×30	9.56	**23.9**	35.9
W14×30	8.99	**22.4**	33.7
W10×30	8.84	**22.1**	33.2
W8×24	**8.57**	**21.4**	**32.1**
W12×26	8.17	**20.4**	30.6
W18×35	8.06	**20.1**	30.2
W10×26	7.50	**18.7**	28.1
W16×31	7.03	**17.5**	26.4
W10×22	**6.10**	**15.2**	**22.9**
W8×21	**5.69**	**14.2**	**21.3**
W14×26	5.54	**13.8**	20.8
W16×26	5.48	**13.7**	20.6
W8×18	**4.66**	**11.6**	**17.5**
W14×22	4.39	**11.0**	16.5
W12×22	3.66	**9.13**	13.7
W10×19	3.35	**8.36**	12.6
W12×19	2.98	**7.44**	11.2
W10×17	**2.80**	**6.99**	**10.5**
W8×15	**2.67**	**6.66**	**10.0**
W10×15	**2.30**	**5.74**	**8.63**
W12×16	2.26	**5.63**	8.46
W8×13	**2.15**	**5.36**	**8.06**
W12×14	1.90	**4.74**	7.13
W10×12[f]	**1.74**	**4.30**	**6.46**
W8×10[f]	**1.66**	**4.07**	**6.12**

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape exceeds compact limit for flexure with $F_y = 50$ ksi.

Table 3–5

W Shapes

Selection by I_y

I_y

Shape	I_y
	in.^{4}
W14×730[h]	**4720**
W36×800[h]	4200
W14×665[h]	**4170**
W14×605[h]	**3680**
W14×550[h]	**3250**
W36×652[h]	3230
W14×500[h]	**2880**
W14×455[h]	**2560**
W40×593[h]	2520
W36×529[h]	**2490**
W14×426[h]	**2360**
W36×487[h]	2250
W14×398[h]	**2170**
W27×539[h]	2110
W40×503[h]	2040
W36×441[h]	1990
W14×370[h]	**1990**
W14×342[h]	**1810**
W36×395[h]	1750
W40×431[h]	1690
W33×387[h]	1620
W14×311[h]	**1610**
W36×361[h]	1570
W30×391[h]	1550
W40×397[h]	1540
W33×354[h]	1460
W14×283[h]	**1440**
W40×372[h]	1420
W36×330	1420
W30×357[h]	1390
W40×362[h]	1380
W27×368[h]	1310
W36×302	1300
W33×318	1290
W14×257	**1290**
W30×326[h]	1240
W40×324	1220
W44×335	1200
W36×282	1200
W12×336[h]	1190
W27×336[h]	1180
W33×291	1160
W24×370[h]	1160
W14×233	**1150**
W30×292	1100
W40×297	1090
W36×262	1090
W27×307[h]	1050
W12×305[h]	1050
W44×290	1040
W40×277	1040
W33×263	1040
W24×335[h]	1030
W14×211	**1030**
W36×247	1010
W30×261	959
W27×281	953
W36×231	940
W12×279[h]	937
W33×241	933
W14×193	**931**
W40×249	926
W44×262	923
W24×306[h]	919
W27×258	859
W30×235	855
W33×221	840
W14×176	**838**
W12×252[h]	828
W24×279[h]	823
W40×392[h]	803
W44×230	796
W40×215	796
W18×311[h]	795
W27×235	769
W30×211	757
W33×201	749
W14×159	**748**
W12×230[h]	742
W24×250	724
W27×217	704
W18×283[h]	704
W40×199	695
W14×145	**677**
W30×191	673
W12×210	664
W24×229	651
W40×331[h]	644
W40×327[h]	640
W18×258[h]	628
W27×194	619
W30×173	598
W12×190	589
W24×207	578
W40×294	562
W18×234[h]	558
W27×178	555
W14×132	**548**
W21×201	542
W24×192	530
W36×256	528
W40×278	521
W12×170	517
W27×161	497
W14×120	**495**
W40×264	493
W18×211	493
W21×182	483
W24×176	479
W36×232	468
W12×152	454
W14×109	**447**
W40×235	444
W27×146	443
W24×162	443
W18×192	440
W21×166	435
W36×210	411
W14×99	**402**
W12×136	398
W24×146	391
W18×175	391
W40×211	390
W21×147	376
W36×194	375

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

I_y

Table 3–5 (continued)
W Shapes
Selection by I_y

Shape	I_y	Shape	I_y	Shape	I_y	Shape	I_y
	in.^{4}		in.^{4}		in.^{4}		in.^{4}
W14×90	**362**	**W12×65**	**174**	**W8×48**	**60.9**	**W8×28**	**21.7**
W36×182	347	W30×116	164	W18×71	60.3	W21×44	20.7
W18×158	347	W16×89	163	W14×53	57.7	W12×30	20.3
W12×120	345	W27×114	159	W21×62	57.5	W14×30	19.6
W24×131	340	W10×77	154	W12×50	56.3	W18×40	19.1
W21×132	333	W18×76	152	W18×65	54.8		
W40×183	331	W14×82	148			**W8×24**	**18.3**
W36×170	320	W30×108	146	**W10×45**	**53.4**	W12×26	17.3
W18×143	311	W27×102	139	W14×48	51.4	W10×30	16.7
W33×169	310	W16×77	138	W18×60	50.1	W18×35	15.3
W21×122	305	W14×74	134			W10×26	14.1
W12×106	301	W10×68	134	**W12×45**	**50.0**	W16×31	12.4
W24×117	297	W30×99	128				
W36×160	295	W27×94	124	**W8×40**	**49.1**	**W10×22**	**11.4**
W40×167	283	W14×68	121	W21×55	48.4		
W18×130	278	W24×103	119	W14×43	45.2	**W8×21**	**9.77**
W21×111	274	W16×67	119			W16×26	9.59
W33×152	273			**W10×39**	**45.0**	W14×26	8.91
W36×150	270	**W10×60**	**116**	W18×55	44.9		
W12×96	270	W30×90	115	W12×40	44.1	**W8×18**	**7.97**
W24×104	259	W24×94	109	W16×57	43.1	W14×22	7.00
W18×119	253	W14×61	107			W12×22	4.66
W21×101	248			**W8×35**	**42.6**	W10×19	4.29
W33×141	246	**W12×58**	**107**	W18×50	40.1	W12×19	3.76
		W27×84	106	W21×48	38.7		
W12×87	**241**			W16×50	37.2	**W10×17**	**3.56**
W10×112	236	**W10×54**	**103**				
W40×149	229			**W8×31**	**37.1**	**W8×15**	**3.41**
W30×148	227	**W12×53**	**95.8**	W10×33	36.6		
W36×135	225	W24×84	94.4	W24×62	34.5	**W10×15**	**2.89**
W18×106	220			W16×45	32.8	W12×16	2.82
W33×130	218	**W10×49**	**93.4**	W21×57	30.6		
		W21×93	92.9	W24×55	29.1	**W8×13**	**2.73**
W12×79	**216**	W8×67	88.6	W16×40	28.9	W12×14	2.36
W10×100	207	W24×76	82.5	W14×38	26.7		
W18×97	201	W21×83	81.4	W21×50	24.9	**W10×12**	**2.18**
W30×132	196	W8×58	75.1	W16×36	24.5		
		W21×73	70.6	W12×35	24.5	**W8×10**	**2.09**
W12×72	**195**	W24×68	70.4	W14×34	23.3		
W33×118	187	W21×68	64.7	W18×46	22.5		
W16×100	186						
W27×129	184						
W30×124	181						
W10×88	179						
W18×86	175						

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

beam weight, which should be deducted when calculating the maximum uniform load the beam will support. C_b is taken as unity.

When the plotted curve is solid, the W-shape for that curve is the lightest cross-section for a given combination of available flexural strength and unbraced length. When the plotted curve is dashed, a lighter W-shape than that for the plotted curve exists. The plotted curves are arbitrarily terminated at a span-to-depth ratio of 30 in most cases.

L_p is indicated in each curve as a solid dot (•). L_r is indicated in each curve as an open dot (○).

Tables 3–11. C- and MC-Shapes—Plots of Available Moment vs. Unbraced Length

Table 3–11 is similar to Table 3–10, except it covers C- and MC-shapes with F_y = 36 ksi (ASTM A36).

AVAILABLE FLEXURAL STRENGTH OF HSS

Table 3–12. Rectangular HSS—Available Flexural Strength

The available flexural strength is tabulated for rectangular HSS with F_y = 46 ksi (ASTM A500 grade B). For non-compact and slender cross-sections, the tabulated values of M_p/Ω_b and $\phi_b M_p$ have been adjusted to account for the non-compactness or slenderness.

Table 3–13. Square HSS—Available Flexural Strength

Table 3–13 is similar to Table 3–12, except it covers square HSS with F_y = 46 ksi (ASTM A500 grade B).

Table 3–14. Round HSS—Available Flexural Strength

Table 3–14 is similar to Table 3–12, except it covers round HSS with F_y = 42 ksi (ASTM A500 grade B).

Table 3–15. Pipe—Available Flexural Strength

Table 3–15 is similar to Table 3–12, except it covers Pipe with F_y = 35 ksi (ASTM A53 grade B).

STRENGTH OF OTHER FLEXURAL MEMBERS

Tables 3–16 and 3–17. Available Shear Stress in Plate Girders

The available shear stress for plate girders is plotted as a function of a/h and h/t_w in Tables 3–16 (for F_y = 36 ksi) and 3–17 (for F_y = 50 ksi). In part a of each table, tension-field action is neglected. In part b of each table, tension-field action is considered.

Table 3–18. Floor Plates

The recommended maximum uniformly distributed loads are given in Table 3–18 based upon simple-span bending between supports. Table 3–18a is for deflection-controlled applications and should be used with the appropriate serviceability load combinations. The tabulated values correspond to a maximum deflection of $L/100$. Table 3–18b is for flexural-strength-controlled applications and should be used with LRFD or ASD load combinations. The tabulated values correspond to a maximum bending stress of 24 ksi in LRFD and 16 ksi in ASD.

COMPOSITE BEAM SELECTION TABLES

Table 3–19. Composite W-Shapes

The available flexural strength is tabulated for W-shapes with F_y = 50 ksi (ASTM A992). The values tabulated are independent of the specific concrete flange properties. The designer can then select an appropriate combination of concrete strength and slab geometry.

The location of the plastic neutral axis (PNA) is uniquely determined by the horizontal shear force ΣQ_n at the interface between the steel section and the concrete slab. With the knowledge of the location of the PNA and the distance to the centroid of the concrete flange force ΣQ_n, the available flexural strength can be computed.

Available flexural strengths are tabulated for plastic neutral axis (PNA) locations at the seven locations shown. Five of these PNA locations are in the beam flange. The seventh PNA location is computed at the point where ΣQ_n equals $0.25F_yA_s$, and the sixth PNA location is at the midpoint between five and seven. Use of beams with a PNA below location seven is discouraged.

Table 3–19 can be used to design a composite beam by entering with a required flexural strength and determining the corresponding required ΣQ_n. Alternatively, Table 3–19 can be used to check the flexural strength of a composite beam by selecting a valid value of ΣQ_n, using Table 3–21. With the effective width of the concrete flange b determined per AISC Specification Section I3.1a, the appropriate value of the distance from concrete flange force to beam top flange $Y2$ can be determined as

$$Y_2 = Y_{con} - \frac{a}{2}$$

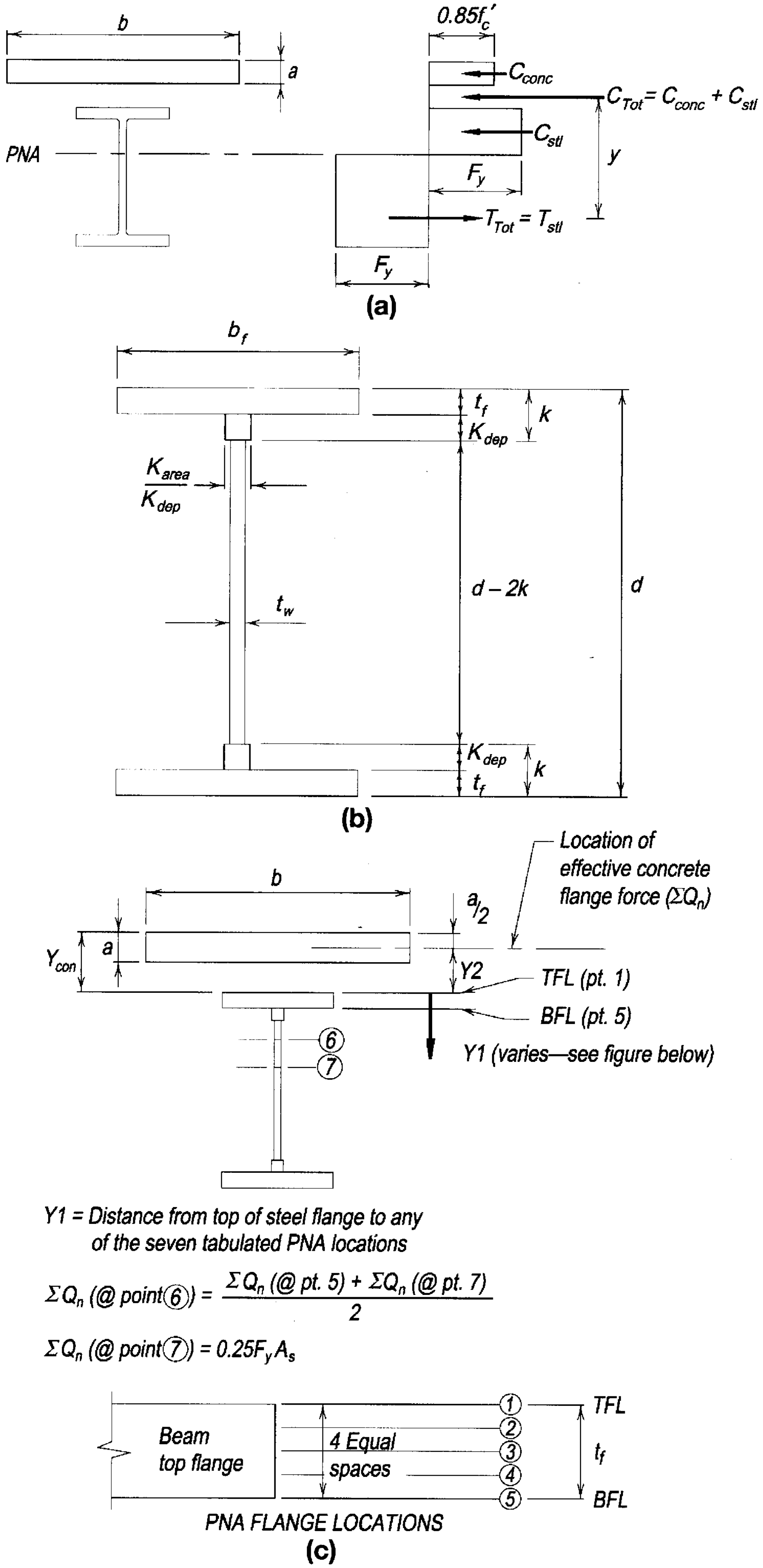
where

Y_{con} = distance from top of steel beam to top of concrete, in.

$$a = \frac{\Sigma Q_n}{0.85 f'_c b}$$

and the available flexural strength, $\phi_b M_n$ or M_n/Ω_b, can then be determined from Table 3–19. Values for the distance from the PNA to the beam top flange $Y1$ are also tabulated for convenience. The parameters $Y1$ and $Y2$ are illustrated in Figure 3–3. Note that the model of the steel beam used in the calculation of the available strength assumes that:

A_s = cross-sectional area of the steel section, in.^{2}
A_f = flange area = $b_f \times t_f$, in.^{2}
A_w = web area = $(d - 2k)t_w$, in.^{2}
$K_{dep} = k - t_f$, in.
$K_{area} = (A_s - 2A_f - A_w)/2$, in.^{2}



Y1 = Distance from top of steel flange to any of the seven tabulated PNA locations

$$\Sigma Q_n\ (@\ \text{point}⑥) = \frac{\Sigma Q_n\ (@\ \text{pt. }5) + \Sigma Q_n\ (@\ \text{pt. }7)}{2}$$

$$\Sigma Q_n\ (@\ \text{point}⑦) = 0.25F_yA_s$$

Figure 3–3. Strength design models for composite beams.

The beam end reactions for symmetrically loaded composite W-shapes can be determined as follows. When the properties of the composite concrete flange have been computed, ΣQ_n can be taken as the smaller of nQ_n, F_yA_s or $0.85f_c'A_c$. With $Y2$ taken as the distance from the top of the steel beam to the top of the concrete slab less $[\Sigma Q_n/(0.85f_c'b)]/2$, the value of available flexural strength, ϕ_bM_n or M_n/Ω_b, can be selected from Table 3-19 and the beam end reaction, R_u or R_a, can be determined as:

LRFD	**ASD**
$R_u = \phi_b M_n \dfrac{C_c}{L}$	$R_a = \dfrac{M_n}{\Omega_b}\dfrac{C_c}{L}$

where

C_c = coefficient from Figure 3–4

L = span length, ft

This value is then useful to check the shear strength.

When the properties of the composite concrete flange have not been computed, a conservative value for ΣQ_n can be taken as the smaller of F_sA_s or nQ_n, where n is the number of

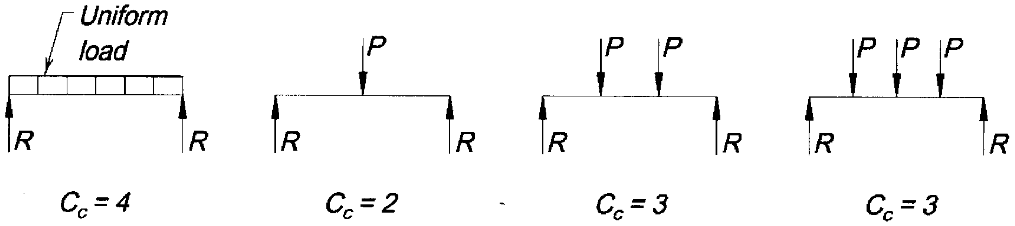


Figure 3–4. Coefficients for use in determining composite simple-beam end reactions.

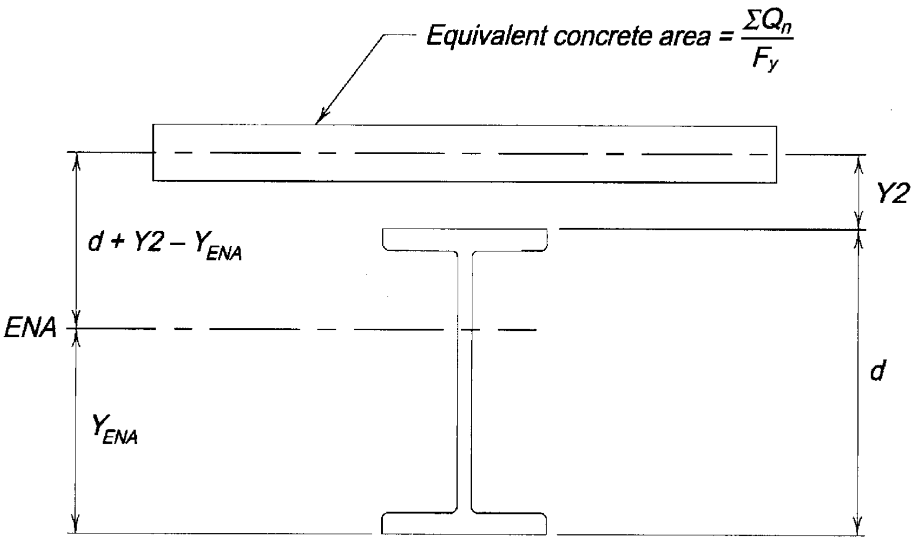


Figure 3–5. Deflection design model for composite beams.

shear stud connectors between the beam end and the point of maximum moment. In this case, *Y*2 is equal to the distance from the top of the steel beam to the top of the concrete slab.

Table 3–20. Lower-Bound Elastic Moments of Inertia

The lower-bound elastic moment of inertia of a composite beam can be used to calculate deflection. If calculated deflections using the lower-bound moment of inertia are acceptable, a more complete elastic analysis of the composite section can be avoided. The lower-bound elastic moment of inertia is based upon the area of the beam and an equivalent concrete area equal to $\Sigma Q_n/F_y$ as illustrated in Figure 3–5. The analysis includes only the horizontal shear force transferred by the shear connectors supplied. Thus, only the portion of the concrete flange used to balance ΣQ_n is included in the determination of the lower-bound moment of inertia.

The value for the lower bound moment of inertia can be calculated as illustrated in AISC Commentary Section I3.2a and Section I3.2b. The lower bound moment of inertia, therefore, is the moment of inertia of the cross-section at the required strength level. This is smaller than the corresponding moment of inertia at the service load where deflection is calculated.

Table 3–21. Nominal Horizontal Shear for One Shear Stud, Q_n

The nominal shear strength of stud shear connectors is given in Table 3–21, in accordance with AISC Specification Chapter I. Nominal horizontal shear strength values are presented based upon the position of the stud, profile of the deck, and orientation of the deck relative to the stud.

BEAM DIAGRAMS AND FORMULAS

Table 3–22a. Concentrated Load Equivalents

Concentrated load equivalents are given in Table 3–22a for beams with various support conditions and loading characteristics.

Table 3–22b. Cantilevered Beams

Coefficients are provided in Table 3–22b for cantilevered beams with various support conditions and loading characteristics.

Table 3-22c. Continuous Beams

Coefficients are provided in Table 3–22c for continuous beams with various support conditions and loading characteristics.

Table 3–23. Shears, Moments, and Deflections

Shears, moments and deflections are given in Table 3–23 for beams with various support conditions and loading characteristics.

F_y = 50 ksi

Table 3–6
Maximum Total Uniform Load, kips
W Shapes

W44

Shape		W44×							
		335		290		262		230^v	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	17		2710						
	18	1800	2700	1510	2260	1360	2040		
	19	1700	2560	1480	2230	1330	2010		
	20	1620	2430	1410	2120	1270	1910	1090	1650
	21	1540	2310	1340	2010	1210	1810	1050	1570
	22	1470	2210	1280	1920	1150	1730	998	1500
	23	1410	2110	1220	1840	1100	1660	955	1430
	24	1350	2030	1170	1760	1060	1590	915	1370
	25	1290	1940	1130	1690	1010	1520	878	1320
	26	1240	1870	1080	1630	975	1470	844	1270
	27	1200	1800	1040	1570	939	1410	813	1220
	28	1150	1740	1010	1510	905	1360	784	1180
	29	1120	1680	970	1460	874	1310	757	1140
	30	1080	1620	938	1410	845	1270	732	1100
	32	1010	1520	879	1320	792	1190	686	1030
	34	951	1430	828	1240	746	1120	646	971
	36	898	1350	782	1180	704	1060	610	917
	38	851	1280	741	1110	667	1000	578	868
	40	808	1220	704	1060	634	953	549	825
	42	770	1160	670	1010	604	907	523	786
	44	735	1100	640	961	576	866	499	750
	46	703	1060	612	920	551	828	477	717
	48	674	1010	586	881	528	794	457	687
	50	647	972	563	846	507	762	439	660
	52	622	935	541	813	487	733	422	635
	54	599	900	521	783	469	706	407	611
	56	577	868	503	755	453	680	392	589
	58	558	838	485	729	437	657	379	569
	60	539	810	469	705	422	635	366	550
	62	522	784	454	682	409	615	354	532
	64	505	759	440	661	396	595	343	516
	66	490	736	426	641	384	577	333	500
	68	476	715	414	622	373	560	323	485
	70	462	694	402	604	362	544	314	471
	72	449	675	391	588	352	529	305	458
Beam Properties									
W_c/Ω_b	$\phi_b W_c$, kip-ft	32300	48600	28100	42300	25300	38100	22000	33000
M_p/Ω_b	$\phi_b M_p$, kip-ft	4040	6080	3520	5290	3170	4760	2740	4130
M_r/Ω_b	$\phi_b M_r$, kip-ft	2460	3700	2170	3260	1940	2910	1700	2550
BF	BF, kips	59.6	89.6	54.9	82.5	52.5	79.0	47.1	70.9
V_n/Ω_v	$\phi_v V_n$, kips	902	1350	755	1130	680	1020	547	823
Z_x, in.^{3}		1620		1410		1270		1100	
L_p, ft		12.3		12.3		12.3		12.1	
L_r, ft		38.8		37.0		35.7		34.4	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

^{v} Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

Note: For beams laterally unsupported, see Table 3–10.

Available strength tabulated above heavy line is limited by available shear strength.

W40

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

$F_y = 50$ ksi

Shape		W40×											
		593[h]		503[h]		431[h]		397[h]		392[h]		372[h]	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	14									2350	3530		
	15									2280	3420		
	16									2130	3210		
	17	3080	4620	2580	3870	2210	3320			2010	3020	1890	2830
	18	3060	4600	2560	3850	2170	3270	2000	3000	1900	2850	1860	2800
	19	2900	4360	2430	3650	2060	3090	1890	2840	1800	2700	1760	2650
	20	2750	4140	2310	3470	1960	2940	1800	2700	1710	2570	1680	2520
	21	2620	3940	2200	3300	1860	2800	1710	2570	1630	2440	1600	2400
	22	2500	3760	2100	3150	1780	2670	1630	2450	1550	2330	1520	2290
	23	2400	3600	2000	3010	1700	2560	1560	2350	1480	2230	1460	2190
	24	2300	3450	1920	2890	1630	2450	1500	2250	1420	2140	1400	2100
	25	2200	3310	1840	2770	1560	2350	1440	2160	1370	2050	1340	2020
	26	2120	3180	1770	2670	1500	2260	1380	2080	1310	1970	1290	1940
	27	2040	3070	1710	2570	1450	2180	1330	2000	1260	1900	1240	1870
	28	1970	2960	1650	2480	1400	2100	1280	1930	1220	1830	1200	1800
	29	1900	2860	1590	2390	1350	2030	1240	1860	1180	1770	1160	1740
	30	1840	2760	1540	2310	1300	1960	1200	1800	1140	1710	1120	1680
	32	1720	2590	1440	2170	1220	1840	1120	1690	1070	1600	1050	1580
	34	1620	2440	1360	2040	1150	1730	1060	1590	1000	1510	986	1480
	36	1530	2300	1280	1920	1090	1630	998	1500	948	1430	931	1400
	38	1450	2180	1210	1820	1030	1550	945	1420	898	1350	882	1330
	40	1380	2070	1150	1730	978	1470	898	1350	853	1280	838	1260
	42	1310	1970	1100	1650	931	1400	855	1290	813	1220	798	1200
	44	1250	1880	1050	1580	889	1340	817	1230	776	1170	762	1150
	46	1200	1800	1000	1510	850	1280	781	1170	742	1120	729	1100
	48	1150	1730	961	1440	815	1230	749	1130	711	1070	699	1050
	50	1100	1660	922	1390	782	1180	719	1080	683	1030	671	1010
	52	1060	1590	887	1330	752	1130	691	1040	656	987	645	969
	54	1020	1530	854	1280	724	1090	665	1000	632	950	621	933
	56	984	1480	823	1240	699	1050	642	964	609	916	599	900
	58	950	1430	795	1190	675	1010	619	931	588	884	578	869
	60	918	1380	768	1160	652	980	599	900	569	855	559	840
	62	889	1340	744	1120	631	948	579	871	551	827	541	813
	64	861	1290	720	1080	611	919	561	844	533	802	524	788
	66	835	1250	699	1050	593	891	544	818	517	777	508	764
	68	810	1220	678	1020	575	865	528	794	502	754	493	741
	70	787	1180	659	990	559	840	513	771	488	733	479	720
	72	765	1150	640	962	543	817	499	750	474	713	466	700
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	55100	82800	46100	69300	39100	58800	35900	54000	34100	51300	33500	50400
M_p/Ω_b	$\phi_b M_p$, kip-ft	6890	10400	5760	8660	4890	7350	4490	6750	4270	6410	4190	6300
M_r/Ω_b	$\phi_b M_r$, kip-ft	4090	6140	3460	5200	2950	4440	2720	4100	2510	3780	2550	3830
BF	BF, kips	55.5	83.5	54.7	82.2	53.6	80.6	52.3	78.7	60.4	90.8	51.6	77.6
V_n/Ω_v	$\phi_v V_n$, kips	1540	2310	1290	1940	1110	1660	999	1500	1180	1760	943	1410
Z_x, in.^{3}		2760		2310		1960		1800		1710		1680	
L_p, ft		13.4		13.1		12.9		12.9		9.33		12.7	
L_r, ft		63.8		55.3		49.0		46.6		38.3		44.5	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W40

Shape		W40×											
		362^h		331^h		327^h		324^h		297		294	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	14			1990	2990	1930	2890					1710	2570
	15			1900	2860	1880	2820					1690	2540
	16			1780	2680	1760	2640					1580	2380
	17			1680	2520	1660	2490			1480		1490	2240
	18	1820	2720	1590	2380	1560	2350	1610	2410	1470	2220	1410	2120
	19	1720	2590	1500	2260	1480	2230	1530	2310	1400	2100	1330	2010
	20	1640	2460	1430	2150	1410	2120	1460	2190	1330	2000	1270	1910
	21	1560	2340	1360	2040	1340	2010	1390	2090	1260	1900	1210	1810
	22	1490	2240	1300	1950	1280	1920	1320	1990	1210	1810	1150	1730
	23	1420	2140	1240	1870	1220	1840	1270	1900	1150	1730	1100	1660
	24	1360	2050	1190	1790	1170	1760	1210	1820	1110	1660	1060	1590
	25	1310	1970	1140	1720	1130	1690	1170	1750	1060	1600	1010	1520
	26	1260	1890	1100	1650	1080	1630	1120	1680	1020	1530	975	1470
	27	1210	1820	1060	1590	1040	1570	1080	1620	983	1480	939	1410
	28	1170	1760	1020	1530	1010	1510	1040	1560	948	1430	905	1360
	29	1130	1700	984	1480	970	1460	1000	1510	915	1380	874	1310
	30	1090	1640	951	1430	938	1410	971	1460	885	1330	845	1270
	32	1020	1540	892	1340	879	1320	911	1370	830	1250	792	1190
	34	963	1450	839	1260	828	1240	857	1290	781	1170	746	1120
	36	909	1370	793	1190	782	1180	809	1220	737	1110	704	1060
	38	861	1290	751	1130	741	1110	767	1150	699	1050	667	1000
	40	818	1230	714	1070	704	1060	729	1100	664	998	634	953
	42	779	1170	680	1020	670	1010	694	1040	632	950	604	907
	44	744	1120	649	975	640	961	662	995	603	907	576	866
	46	712	1070	620	933	612	920	634	952	577	867	551	828
	48	682	1020	595	894	586	881	607	912	553	831	528	794
	50	655	984	571	858	563	846	583	876	531	798	507	762
	52	630	946	549	825	541	813	560	842	511	767	487	733
	54	606	911	529	794	521	783	540	811	492	739	469	706
	56	585	879	510	766	503	755	520	782	474	713	453	680
	58	564	848	492	740	485	729	502	755	458	688	437	657
	60	546	820	476	715	469	705	486	730	442	665	422	635
	62	528	794	460	692	454	682	470	706	428	644	409	615
	64	511	769	446	670	440	661	455	684	415	623	396	595
	66	496	745	432	650	426	641	442	664	402	605	384	577
	68	481	724	420	631	414	622	429	644	390	587	373	560
	70	468	703	408	613	402	604	416	626	379	570	362	544
	72	455	683	396	596	391	588	405	608	369	554	352	529
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	32700	49200	28500	42900	28100	42300	29100	43800	26500	39900	25300	38100
M_p/Ω_b	$\phi_b M_p$, kip-ft	4090	6150	3570	5360	3520	5290	3640	5480	3320	4990	3170	4760
M_r/Ω_b	$\phi_b M_r$, kip-ft	2480	3730	2110	3180	2100	3150	2240	3360	2040	3070	1890	2840
BF	BF, kips	51.5	77.4	59.0	88.7	58.0	87.2	49.1	73.8	47.3	71.1	57.0	85.7
V_n/Ω_v	$\phi_v V_n$, kips	908	1360	995	1490	963	1440	803	1200	741	1110	856	1280
Z_x, in.^{3}		1640		1430		1410		1460		1330		1270	
L_p, ft		12.7		9.08		9.11		12.6		12.5		9.01	
L_r, ft		44.0		33.7		33.6		41.3		39.4		31.5	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W40

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W40×											
		278		277		264		249		235		215	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	14	1650	2470			1540	2300						
	15	1580	2380			1500	2260			1320	1980		
	16	1480	2230			1410	2120			1260	1890		
	17	1400	2100			1330	1990			1190	1780		
	18	1320	1980	1320	1980	1250	1880			1120	1680		
	19	1250	1880	1310	1970	1190	1780	1180	1770	1060	1590	1010	1520
	20	1190	1790	1250	1880	1130	1700	1120	1680	1010	1520	962	1450
	21	1130	1700	1190	1790	1070	1610	1060	1600	960	1440	916	1380
	22	1080	1620	1130	1700	1030	1540	1020	1530	916	1380	875	1310
	23	1030	1550	1080	1630	981	1470	972	1460	877	1320	837	1260
	24	990	1490	1040	1560	940	1410	931	1400	840	1260	802	1200
	25	950	1430	998	1500	902	1360	894	1340	806	1210	770	1160
	26	914	1370	960	1440	867	1300	860	1290	775	1170	740	1110
	27	880	1320	924	1390	835	1260	828	1240	747	1120	713	1070
	28	848	1280	891	1340	806	1210	798	1200	720	1080	687	1030
	29	819	1230	860	1290	778	1170	771	1160	695	1040	664	997
	30	792	1190	832	1250	752	1130	745	1120	672	1010	641	964
	32	742	1120	780	1170	705	1060	699	1050	630	947	601	904
	34	699	1050	734	1100	663	997	658	988	593	891	566	851
	36	660	992	693	1040	627	942	621	933	560	842	534	803
	38	625	939	657	987	594	892	588	884	531	797	506	761
	40	594	893	624	938	564	848	559	840	504	758	481	723
	42	566	850	594	893	537	807	532	800	480	721	458	689
	44	540	811	567	852	513	770	508	764	458	689	437	657
	46	516	776	542	815	490	737	486	730	438	659	418	629
	48	495	744	520	781	470	706	466	700	420	631	401	602
	50	475	714	499	750	451	678	447	672	403	606	385	578
	52	457	687	480	721	434	652	430	646	388	583	370	556
	54	440	661	462	694	418	628	414	622	373	561	356	536
	56	424	638	446	670	403	605	399	600	360	541	344	516
	58	410	616	430	647	389	584	385	579	348	522	332	499
	60	396	595	416	625	376	565	373	560	336	505	321	482
	62	383	576	402	605	364	547	361	542	325	489	310	466
	64	371	558	390	586	352	530	349	525	315	473	301	452
	66	360	541	378	568	342	514	339	509	305	459	292	438
	68	349	525	367	551	332	499	329	494	296	446	283	425
	70	339	510	356	536	322	484	319	480	288	433	275	413
	72	330	496	347	521	313	471	310	467	280	421	267	402
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	23800	35700	25000	37500	22600	33900	22400	33600	20200	30300	19200	28900
M_p/Ω_b	$\phi_b M_p$, kip-ft	2970	4460	3120	4690	2820	4240	2790	4200	2520	3790	2410	3620
M_r/Ω_b	$\phi_b M_r$, kip-ft	1780	2680	1920	2890	1700	2550	1730	2610	1530	2300	1500	2250
BF	BF, kips	55.2	82.9	45.8	68.9	54.1	81.4	43.0	64.7	51.0	76.7	39.2	58.9
V_n/Ω_v	$\phi_v V_n$, kips	823	1230	659	988	768	1150	591	886	659	988	507	760
Z_x, in.^{3}		1190		1250		1130		1120		1010		964	
L_p, ft		8.90		12.6		8.90		12.5		8.97		12.5	
L_r, ft		30.4		38.8		29.7		37.2		28.4		35.6	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W40–W36

Shape		W40×										W36×	
		211		199		183		167		149^v		800^h	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	13							1000	1510	865	1300		
	14							988	1490	853	1280		
	15	1180	1770			1010	1520	922	1390	796	1200		
	16	1130	1700			966	1450	865	1300	746	1120		
	17	1060	1600	1010	1510	909	1370	814	1220	702	1060		
	18	1000	1510	964	1450	858	1290	768	1160	663	997	4050	6080
	19	952	1430	913	1370	813	1220	728	1090	628	944	3830	5760
	20	904	1360	867	1300	772	1160	692	1040	597	897	3640	5480
	21	861	1290	826	1240	736	1110	659	990	568	854	3470	5210
	22	822	1240	788	1190	702	1060	629	945	543	815	3310	4980
	23	786	1180	754	1130	672	1010	601	904	519	780	3170	4760
	24	753	1130	723	1090	644	968	576	866	497	747	3040	4560
	25	723	1090	694	1040	618	929	553	832	477	718	2910	4380
	26	696	1050	667	1000	594	893	532	800	459	690	2800	4210
	27	670	1010	642	966	572	860	512	770	442	664	2700	4060
	28	646	971	619	931	552	829	494	743	426	641	2600	3910
	29	624	937	598	899	533	801	477	717	412	619	2510	3780
	30	603	906	578	869	515	774	461	693	398	598	2430	3650
	32	565	849	542	815	483	726	432	650	373	561	2280	3420
	34	532	799	510	767	454	683	407	611	351	528	2140	3220
	36	502	755	482	724	429	645	384	578	332	498	2020	3040
	38	476	715	456	686	407	611	364	547	314	472	1920	2880
	40	452	680	434	652	386	581	346	520	298	449	1820	2740
	42	431	647	413	621	368	553	329	495	284	427	1730	2610
	44	411	618	394	593	351	528	314	473	271	408	1660	2490
	46	393	591	377	567	336	505	301	452	259	390	1580	2380
	48	377	566	361	543	322	484	288	433	249	374	1520	2280
	50	362	544	347	521	309	464	277	416	239	359	1460	2190
	52	348	523	334	501	297	447	266	400	230	345	1400	2110
	54	335	503	321	483	286	430	256	385	221	332	1350	2030
	56	323	485	310	466	276	415	247	371	213	320	1300	1960
	58	312	469	299	449	266	400	238	358	206	309	1260	1890
	60	301	453	289	435	257	387	231	347	199	299	1210	1830
	62	292	438	280	420	249	375	223	335	193	289	1180	1770
	64	283	425	271	407	241	363	216	325	187	280	1140	1710
	66	274	412	263	395	234	352	210	315	181	272	1100	1660
	68	266	400	255	383	227	341	203	306	176	264	1070	1610
	70	258	388	248	372	221	332	198	297	171	256	1040	1560
	72	251	377	241	362	215	323	192	289	166	249	1010	1520
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	18100	27200	17300	26100	15400	23200	13800	20800	11900	17900	72900	110000
M_p/Ω_b	$\phi_b M_p$, kip-ft	2260	3400	2170	3260	1930	2900	1730	2600	1490	2240	9110	13700
M_r/Ω_b	$\phi_b M_r$, kip-ft	1370	2060	1340	2020	1180	1770	1050	1580	896	1350	5310	7980
BF	BF, kips	48.5	73.0	37.2	55.9	44.1	66.3	41.8	62.9	38.6	58.0	47.5	71.4
V_n/Ω_v	$\phi_v V_n$, kips	591	886	503	754	507	760	502	753	432	650	2030	3040
Z_x, in.^{3}		906		869		774		693		598		3650	
L_p, ft		8.87		12.2		8.80		8.48		8.09		14.9	
L_r, ft		27.2		34.3		25.9		24.8		23.5		94.8	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W36

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W36×											
		652[h]		529[h]		487[h]		441[h]		395[h]		361[h]	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	18	3230	4850	2560	3840	2360	3540	2110	3170	1870	2810	1700	2550
	19	3060	4590	2450	3680	2240	3360	2010	3020	1800	2700	1630	2450
	20	2900	4370	2330	3500	2130	3200	1910	2870	1710	2570	1550	2330
	21	2770	4160	2210	3330	2020	3040	1820	2730	1630	2440	1470	2210
	22	2640	3970	2110	3180	1930	2900	1730	2600	1550	2330	1410	2110
	23	2530	3800	2020	3040	1850	2780	1660	2490	1480	2230	1350	2020
	24	2420	3640	1940	2910	1770	2660	1590	2390	1420	2140	1290	1940
	25	2320	3490	1860	2800	1700	2560	1520	2290	1370	2050	1240	1860
	26	2230	3360	1790	2690	1640	2460	1470	2200	1310	1970	1190	1790
	27	2150	3230	1720	2590	1570	2370	1410	2120	1260	1900	1150	1720
	28	2070	3120	1660	2500	1520	2280	1360	2050	1220	1830	1100	1660
	29	2000	3010	1600	2410	1470	2200	1310	1980	1180	1770	1070	1600
	30	1940	2910	1550	2330	1420	2130	1270	1910	1140	1710	1030	1550
	32	1820	2730	1450	2180	1330	2000	1190	1790	1070	1600	967	1450
	34	1710	2570	1370	2060	1250	1880	1120	1690	1000	1510	910	1370
	36	1610	2430	1290	1940	1180	1780	1060	1590	948	1430	859	1290
	38	1530	2300	1220	1840	1120	1680	1000	1510	898	1350	814	1220
	40	1450	2180	1160	1750	1060	1600	953	1430	853	1280	773	1160
	42	1380	2080	1110	1660	1010	1520	908	1360	813	1220	737	1110
	44	1320	1980	1060	1590	966	1450	866	1300	776	1170	703	1060
	46	1260	1900	1010	1520	924	1390	829	1250	742	1120	673	1010
	48	1210	1820	969	1460	886	1330	794	1190	711	1070	645	969
	50	1160	1750	930	1400	850	1280	762	1150	683	1030	619	930
	52	1120	1680	894	1340	818	1230	733	1100	656	987	595	894
	54	1080	1620	861	1290	787	1180	706	1060	632	950	573	861
	56	1040	1560	830	1250	759	1140	681	1020	609	916	552	830
	58	1000	1510	802	1210	733	1100	657	988	588	884	533	802
	60	968	1460	775	1170	709	1070	635	955	569	855	516	775
	62	937	1410	750	1130	686	1030	615	924	551	827	499	750
	64	908	1360	727	1090	664	998	596	895	533	802	483	727
	66	880	1320	705	1060	644	968	578	868	517	777	469	705
	68	854	1280	684	1030	625	940	561	843	502	754	455	684
	70	830	1250	664	999	607	913	545	819	488	733	442	664
	72	807	1210	646	971	590	888	529	796	474	713	430	646
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	58100	87300	46500	69900	42500	63900	38100	57300	34100	51300	30900	46500
M_p/Ω_b	$\phi_b M_p$, kip-ft	7260	10900	5810	8740	5310	7990	4770	7160	4270	6410	3870	5810
M_r/Ω_b	$\phi_b M_r$, kip-ft	4300	6460	3480	5220	3200	4800	2880	4330	2600	3910	2360	3540
BF	BF, kips	46.8	70.4	46.5	70.0	46.1	69.3	45.2	68.0	44.7	67.1	43.7	65.7
V_n/Ω_v	$\phi_v V_n$, kips	1620	2430	1280	1920	1180	1770	1060	1590	937	1410	851	1280
Z_x, in.^{3}		2910		2330		2130		1910		1710		1550	
L_p, ft		14.5		14.1		14.0		13.8		13.7		13.6	
L_r, ft		77.8		64.4		60.0		55.5		51.0		48.1	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W36

Shape		W36×											
		330		302		282		262		247		231	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	17							1240	1860	1170	1760	1110	1660
	18	1540	2310	1410	2120	1310	1970	1220	1830	1140	1720	1070	1610
	19	1480	2230	1340	2020	1250	1880	1160	1740	1080	1630	1010	1520
	20	1410	2120	1280	1920	1190	1790	1100	1650	1030	1550	961	1440
	21	1340	2010	1220	1830	1130	1700	1050	1570	979	1470	915	1380
	22	1280	1920	1160	1750	1080	1620	998	1500	934	1400	874	1310
	23	2020	1220	1840	1110	1670	1030	1550	955	1430	894	1340	836
	24	1170	1760	1060	1600	990	1490	915	1370	857	1290	801	1200
	25	1130	1690	1020	1540	950	1430	878	1320	822	1240	769	1160
	26	1080	1630	983	1480	914	1370	844	1270	791	1190	739	1110
	27	1040	1570	946	1420	880	1320	813	1220	761	1140	712	1070
	28	1010	1510	912	1370	848	1280	784	1180	734	1100	686	1030
	29	970	1460	881	1320	819	1230	757	1140	709	1070	663	996
	30	938	1410	852	1280	792	1190	732	1100	685	1030	641	963
	32	879	1320	798	1200	742	1120	686	1030	642	966	601	903
	34	828	1240	751	1130	699	1050	646	971	605	909	565	850
	36	782	1180	710	1070	660	992	610	917	571	858	534	803
	38	741	1110	672	1010	625	939	578	868	541	813	506	760
	40	704	1060	639	960	594	893	549	825	514	773	481	722
	42	670	1010	608	914	566	850	523	786	489	736	458	688
	44	640	961	581	873	540	811	499	750	467	702	437	657
	46	612	920	555	835	516	776	477	717	447	672	418	628
	48	586	881	532	800	495	744	457	687	428	644	400	602
	50	563	846	511	768	475	714	439	660	411	618	384	578
	52	541	813	491	738	457	687	422	635	395	594	370	556
	54	521	783	473	711	440	661	407	611	381	572	356	535
	56	503	755	456	686	424	638	392	589	367	552	343	516
	58	485	729	440	662	410	616	379	569	354	533	331	498
	60	469	705	426	640	396	595	366	550	343	515	320	482
	62	454	682	412	619	383	576	354	532	332	498	310	466
	64	440	661	399	600	371	558	343	516	321	483	300	451
	66	426	641	387	582	360	541	333	500	311	468	291	438
	68	414	622	376	565	349	525	323	485	302	454	283	425
	70	402	604	365	549	339	510	314	471	294	441	275	413
	72	391	588	355	533	330	496	305	458	286	429	267	401
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	28100	42300	25500	38400	23800	35700	22000	33000	20600	30900	19200	28900
M_p/Ω_b	$\phi_b M_p$, kip-ft	3520	5290	3190	4800	2970	4460	2740	4130	2570	3860	2400	3610
M_r/Ω_b	$\phi_b M_r$, kip-ft	2170	3260	1970	2970	1830	2760	1700	2550	1590	2400	1490	2240
BF	BF, kips	42.3	63.6	40.5	60.9	39.4	59.2	38.4	57.7	37.1	55.8	35.8	53.7
V_n/Ω_v	$\phi_v V_n$, kips	768	1150	706	1060	657	985	619	929	587	880	555	832
Z_x, in.^{3}		1410		1280		1190		1100		1030		963	
L_p, ft		13.5		13.5		13.4		13.3		13.2		13.1	
L_r, ft		45.5		43.6		42.2		40.6		39.5		38.6	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W36

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W36×											
		256		232		210		194		182		170	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	13					1220	1830	1120	1670	1050	1580	984	1480
	14	1440	2160	1290	1940	1190	1790	1090	1640	1020	1540	952	1430
	15	1380	2080	1250	1870	1110	1670	1020	1530	955	1440	889	1340
	16	1300	1950	1170	1760	1040	1560	957	1440	896	1350	833	1250
	17	1220	1840	1100	1650	978	1470	901	1350	843	1270	784	1180
	18	1150	1730	1040	1560	924	1390	851	1280	796	1200	741	1110
	19	1090	1640	983	1480	875	1320	806	1210	754	1130	702	1050
	20	1040	1560	934	1400	831	1250	765	1150	717	1080	667	1000
	21	988	1490	890	1340	792	1190	729	1100	682	1030	635	954
	22	944	1420	849	1280	756	1140	696	1050	651	979	606	911
	23	903	1360	812	1220	723	1090	666	1000	623	937	580	871
	24	865	1300	778	1170	693	1040	638	959	597	898	556	835
	25	830	1250	747	1120	665	1000	612	920	573	862	533	802
	26	798	1200	719	1080	639	961	589	885	551	828	513	771
	27	769	1160	692	1040	616	926	567	852	531	798	494	742
	28	741	1110	667	1000	594	893	547	822	512	769	476	716
	29	716	1080	644	968	573	862	528	793	494	743	460	691
	30	692	1040	623	936	554	833	510	767	478	718	444	668
	32	649	975	584	878	520	781	478	719	448	673	417	626
	34	611	918	549	826	489	735	450	677	422	634	392	589
	36	577	867	519	780	462	694	425	639	398	598	370	557
	38	546	821	492	739	438	658	403	606	377	567	351	527
	40	519	780	467	702	416	625	383	575	358	539	333	501
	42	494	743	445	669	396	595	365	548	341	513	317	477
	44	472	709	425	638	378	568	348	523	326	490	303	455
	46	451	678	406	610	361	543	333	500	312	468	290	436
	48	432	650	389	585	346	521	319	479	299	449	278	418
	50	415	624	374	562	333	500	306	460	287	431	267	401
	52	399	600	359	540	320	481	294	443	276	414	256	385
	54	384	578	346	520	308	463	284	426	265	399	247	371
	56	371	557	334	501	297	446	273	411	256	385	238	358
	58	358	538	322	484	287	431	264	397	247	371	230	346
	60	346	520	311	468	277	417	255	384	239	359	222	334
	62	335	503	301	453	268	403	247	371	231	347	215	323
	64	324	488	292	439	260	390	239	360	224	337	208	313
	66	315	473	283	425	252	379	232	349	217	326	202	304
	68	305	459	275	413	245	368	225	338	211	317	196	295
	70	297	446	267	401	238	357	219	329	205	308	190	286
	72	288	433	259	390	231	347	213	320	199	299	185	278
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	20800	31200	18700	28100	16600	25000	15300	23000	14300	21500	13300	20000
M_p/Ω_b	$\phi_b M_p$, kip-ft	2590	3900	2340	3510	2080	3120	1910	2880	1790	2690	1670	2510
M_r/Ω_b	$\phi_b M_r$, kip-ft	1560	2350	1410	2120	1260	1890	1160	1740	1090	1640	1010	1530
BF	BF, kips	46.5	70.0	44.6	67.1	42.5	63.8	40.6	61.0	39.1	58.8	37.4	56.2
V_n/Ω_v	$\phi_v V_n$, kips	719	1080	646	969	609	914	558	837	527	790	492	738
Z_x, in.^{3}		1040		936		833		767		718		668	
L_p, ft		9.36		9.25		9.11		9.04		9.01		8.94	
L_r, ft		31.5		29.9		28.5		27.6		27.0		26.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi, Ω_v = 1.67, ϕ_v = 0.90.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W36–W33

Shape		W36×						W33×					
		160		150		135^v		387^h		354^h		318	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Span, ft	**12**			**896**									
	13	**936**	1400	**892**	1340	**766**	1150						
	14	**890**	1340	**828**	1250	**726**	1090						
	15	**830**	1250	**773**	1160	**677**	1020						
	16	**778**	1170	**725**	1090	**635**	954						
	17	**733**	1100	**682**	1030	**598**	898	**1810**	2720	**1650**	2470	**1460**	2190
	18	**692**	1040	**644**	968	**564**	848	**1730**	2600	**1570**	2370	**1410**	2120
	19	**656**	985	**610**	917	**535**	804	**1640**	2460	**1490**	2240	**1330**	2010
	20	**623**	936	**580**	872	**508**	764	**1560**	2340	**1420**	2130	**1270**	1910
	21	**593**	891	**552**	830	**484**	727	**1480**	2230	**1350**	2030	**1210**	1810
	22	**566**	851	**527**	792	**462**	694	**1420**	2130	**1290**	1940	**1150**	1730
	23	**542**	814	**504**	758	**442**	664	**1350**	2030	**1230**	1850	**1100**	1660
	24	**519**	780	**483**	726	**423**	636	**1300**	1950	**1180**	1780	**1060**	1590
	25	**498**	749	**464**	697	**406**	611	**1250**	1870	**1130**	1700	**1010**	1520
	26	**479**	720	**446**	670	**391**	587	**1200**	1800	**1090**	1640	**975**	1470
	27	**461**	693	**430**	646	**376**	566	**1150**	1730	**1050**	1580	**939**	1410
	28	**445**	669	**414**	623	**363**	545	**1110**	1670	**1010**	1520	**905**	1360
	29	**429**	646	**400**	601	**350**	527	**1070**	1610	**977**	1470	**874**	1310
	30	**415**	624	**387**	581	**339**	509	**1040**	1560	**945**	1420	**845**	1270
	32	**389**	585	**362**	545	**317**	477	**973**	1460	**886**	1330	**792**	1190
	34	**366**	551	**341**	513	**299**	449	**916**	1380	**834**	1250	**746**	1120
	36	**346**	520	**322**	484	**282**	424	**865**	1300	**787**	1180	**704**	1060
	38	**328**	493	**305**	459	**267**	402	**819**	1230	**746**	1120	**667**	1000
	40	**311**	468	**290**	436	**254**	382	**778**	1170	**709**	1070	**634**	953
	42	**297**	446	**276**	415	**242**	364	**741**	1110	**675**	1010	**604**	907
	44	**283**	425	**264**	396	**231**	347	**708**	1060	**644**	968	**576**	866
	46	**271**	407	**252**	379	**221**	332	**677**	1020	**616**	926	**551**	828
	48	**259**	390	**242**	363	**212**	318	**649**	975	**590**	888	**528**	794
	50	**249**	374	**232**	349	**203**	305	**623**	936	**567**	852	**507**	762
	52	**240**	360	**223**	335	**195**	294	**599**	900	**545**	819	**487**	733
	54	**231**	347	**215**	323	**188**	283	**577**	867	**525**	789	**469**	706
	56	**222**	334	**207**	311	**181**	273	**556**	836	**506**	761	**453**	680
	58	**215**	323	**200**	301	**175**	263	**537**	807	**489**	734	**437**	657
	60	**208**	312	**193**	291	**169**	255	**519**	780	**472**	710	**422**	635
	62	**201**	302	**187**	281	**164**	246	**502**	755	**457**	687	**409**	615
	64	**195**	293	**181**	272	**159**	239	**487**	731	**443**	666	**396**	595
	66	**189**	284	**176**	264	**154**	231	**472**	709	**429**	645	**384**	577
	68	**183**	275	**171**	256	**149**	225	**458**	688	**417**	626	**373**	560
	70	**178**	267	**166**	249	**145**	218	**445**	669	**405**	609	**362**	544
	72	**173**	260	**161**	242	**141**	212	**432**	650	**394**	592	**352**	529
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	**12500**	18700	**11600**	17400	**10200**	15300	**31100**	46800	**28300**	42600	**25300**	38100
M_p/Ω_b	$\phi_b M_p$, kip-ft	**1560**	2340	**1450**	2180	**1270**	1910	**3890**	5850	**3540**	5330	**3170**	4760
M_r/Ω_b	$\phi_b M_r$, kip-ft	**947**	1420	**880**	1320	**767**	1150	**2360**	3540	**2170**	3260	**1940**	2910
BF	BF, kips	**36.0**	54.1	**34.5**	51.8	**31.8**	47.8	**38.4**	57.7	**37.5**	56.4	**36.9**	55.4
V_n/Ω_v	$\phi_v V_n$, kips	**468**	702	**448**	672	**383**	576	**906**	1360	**825**	1240	**731**	1100
Z_x, in.^{3}		624		581		509		1560		1420		1270	
L_p, ft		8.83		8.72		8.41		13.3		13.2		13.1	
L_r, ft		25.8		25.2		24.2		53.3		49.9		46.5	
ASD	**LRFD**	Note: For beams laterally unsupported, see Table 3–10.											
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$	Available strength tabulated above heavy line is limited by available shear strength.											

W33

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

$F_y = 50$ ksi

Shape		W33×											
		291		263		241		221		201		169	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	13											906	1360
	14											897	1350
	15											837	1260
	16					1130	1700	1050	1580	963	1440	785	1180
	17	1340	2010	1200	1800	1100	1660	1010	1510	908	1360	739	1110
	18	1290	1930	1150	1730	1040	1570	950	1430	857	1290	697	1050
	19	1220	1830	1090	1640	987	1480	900	1350	812	1220	661	993
	20	1160	1740	1040	1560	938	1410	855	1290	771	1160	628	944
	21	1100	1660	988	1490	893	1340	815	1220	735	1100	598	899
	22	1050	1580	944	1420	853	1280	778	1170	701	1050	571	858
	23	1010	1510	903	1360	816	1230	744	1120	671	1010	546	820
	24	965	1450	865	1300	782	1180	713	1070	643	966	523	786
	25	926	1390	830	1250	750	1130	684	1030	617	928	502	755
	26	891	1340	798	1200	722	1080	658	989	593	892	483	726
	27	858	1290	769	1160	695	1040	634	952	571	859	465	699
	28	827	1240	741	1110	670	1010	611	918	551	828	448	674
	29	798	1200	716	1080	647	972	590	887	532	800	433	651
	30	772	1160	692	1040	625	940	570	857	514	773	418	629
	32	724	1090	649	975	586	881	535	803	482	725	392	590
	34	681	1020	611	918	552	829	503	756	454	682	369	555
	36	643	967	577	867	521	783	475	714	429	644	349	524
	38	609	916	546	821	494	742	450	677	406	610	330	497
	40	579	870	519	780	469	705	428	643	386	580	314	472
	42	551	829	494	743	447	671	407	612	367	552	299	449
	44	526	791	472	709	426	641	389	584	351	527	285	429
	46	503	757	451	678	408	613	372	559	335	504	273	410
	48	482	725	432	650	391	588	356	536	321	483	262	393
	50	463	696	415	624	375	564	342	514	309	464	251	377
	52	445	669	399	600	361	542	329	494	297	446	241	363
	54	429	644	384	578	347	522	317	476	286	429	232	349
	56	413	621	371	557	335	504	305	459	276	414	224	337
	58	399	600	358	538	323	486	295	443	266	400	216	325
	60	386	580	346	520	313	470	285	429	257	387	209	315
	62	373	561	335	503	303	455	276	415	249	374	202	304
	64	362	544	324	488	293	441	267	402	241	362	196	295
	66	351	527	315	473	284	427	259	390	234	351	190	286
	68	340	512	305	459	276	415	252	378	227	341	185	278
	70	331	497	297	446	268	403	244	367	220	331	179	270
	72	322	483	288	433	261	392	238	357	214	322	174	262
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	23200	34800	20800	31200	18800	28200	17100	25700	15400	23200	12600	18900
M_p/Ω_b	$\phi_b M_p$, kip-ft	2890	4350	2590	3900	2350	3530	2140	3210	1930	2900	1570	2360
M_r/Ω_b	$\phi_b M_r$, kip-ft	1780	2680	1610	2410	1450	2180	1330	1990	1200	1800	959	1440
BF	*BF*, kips	36.0	54.1	34.6	51.9	33.2	49.8	31.8	47.8	30.2	45.3	34.1	51.3
V_n/Ω_v	$\phi_v V_n$, kips	669	1000	601	901	567	851	526	789	482	722	453	680
Z_x, in.^{3}		1160		1040		940		857		773		629	
L_p, ft		13.0		12.9		12.8		12.7		12.6		8.83	
L_r, ft		43.9		41.6		39.7		38.2		36.8		26.7	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

F_y = 50 ksi

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W33–W30

Shape		W33×								W30×			
		152		141		130		118^v		391^h		357^h	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	12			806	1210	768	1150	649	976				
	13	851	1280	789	1190	717	1080	637	958				
	14	797	1200	733	1100	666	1000	592	889				
	15	744	1120	684	1030	621	934	552	830				
	16	697	1050	641	964	583	876	518	778	1810	2710	1630	2440
	17	656	986	603	907	548	824	487	732	1700	2560	1550	2330
	18	620	932	570	857	518	778	460	692	1610	2420	1460	2200
	19	587	883	540	812	491	737	436	655	1520	2290	1390	2080
	20	558	839	513	771	466	701	414	623	1450	2180	1320	1980
	21	531	799	489	734	444	667	394	593	1380	2070	1250	1890
	22	507	762	466	701	424	637	377	566	1320	1980	1200	1800
	23	485	729	446	670	405	609	360	541	1260	1890	1150	1720
	24	465	699	427	642	388	584	345	519	1210	1810	1100	1650
	25	446	671	410	617	373	560	331	498	1160	1740	1050	1580
	26	429	645	395	593	359	539	319	479	1110	1670	1010	1520
	27	413	621	380	571	345	519	307	461	1070	1610	976	1470
	28	398	599	366	551	333	500	296	445	1030	1550	941	1410
	29	385	578	354	532	321	483	286	429	998	1500	909	1370
	30	372	559	342	514	311	467	276	415	965	1450	878	1320
	32	349	524	321	482	291	438	259	389	904	1360	823	1240
	34	328	493	302	454	274	412	244	366	851	1280	775	1160
	36	310	466	285	428	259	389	230	346	804	1210	732	1100
	38	294	441	270	406	245	369	218	328	762	1140	693	1040
	40	279	419	256	386	233	350	207	311	724	1090	659	990
	42	266	399	244	367	222	334	197	296	689	1040	627	943
	44	254	381	233	350	212	318	188	283	658	989	599	900
	46	243	365	223	335	203	305	180	271	629	946	573	861
	48	232	349	214	321	194	292	173	259	603	906	549	825
	50	223	335	205	308	186	280	166	249	579	870	527	792
	52	215	323	197	297	179	269	159	239	557	837	507	762
	54	207	311	190	286	173	259	153	231	536	806	488	733
	56	199	299	183	275	166	250	148	222	517	777	470	707
	58	192	289	177	266	161	242	143	215	499	750	454	683
	60	186	280	171	257	155	234	138	208	482	725	439	660
	62	180	270	165	249	150	226	134	201	467	702	425	639
	64	174	262	160	241	146	219	129	195	452	680	412	619
	66	169	254	155	234	141	212	126	189	439	659	399	600
	68	164	247	151	227	137	206	122	183	426	640	387	582
	70	159	240	147	220	133	200	118	178	413	621	376	566
	72	155	233	142	214	129	195	115	173	402	604	366	550

Beam Properties

W_c/Ω_b	$\phi_b W_c$, kip-ft	11200	16800	10300	15400	9320	14000	8280	12500	28900	43500	26300	39600
M_p/Ω_b	$\phi_b M_p$, kip-ft	1390	2100	1280	1930	1170	1750	1040	1560	3620	5440	3290	4950
M_r/Ω_b	$\phi_b M_r$, kip-ft	851	1280	782	1180	709	1070	627	942	2180	3280	1990	2990
BF	BF, kips	32.0	48.1	30.4	45.8	28.8	43.3	26.7	40.2	31.3	47.1	31.2	47.0
V_n/Ω_v	$\phi_v V_n$, kips	425	638	403	604	384	576	325	488	903	1350	813	1220
Z_x, in.^{3}		559		514		467		415		1450		1320	
L_p, ft		8.72		8.58		8.44		8.19		13.0		12.9	
L_r, ft		25.7		25.0		24.3		23.5		58.8		54.5	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W30

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W30×											
		326[h]		292		261		235		211		191	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Span, ft	**15**									959	1440	871	1310
	16	1480	2220	1310	1960	1180	1760	1040	1560	937	1410	842	1270
	17	1400	2100	1240	1870	1110	1660	994	1490	882	1330	793	1190
	18	1320	1980	1180	1770	1050	1570	939	1410	833	1250	749	1130
	19	1250	1880	1110	1670	991	1490	890	1340	789	1190	709	1070
	20	1190	1790	1060	1590	941	1410	845	1270	750	1130	674	1010
	21	1130	1700	1010	1510	896	1350	805	1210	714	1070	642	964
	22	1080	1620	962	1450	856	1290	768	1160	681	1020	612	920
	23	1030	1550	920	1380	818	1230	735	1100	652	980	586	880
	24	990	1490	882	1330	784	1180	704	1060	625	939	561	844
	25	950	1430	846	1270	753	1130	676	1020	600	901	539	810
	26	914	1370	814	1220	724	1090	650	977	577	867	518	779
	27	880	1320	784	1180	697	1050	626	941	555	834	499	750
	28	848	1280	756	1140	672	1010	604	908	535	805	481	723
	29	819	1230	730	1100	649	976	583	876	517	777	465	698
	30	792	1190	705	1060	627	943	564	847	500	751	449	675
	32	742	1120	661	994	588	884	528	794	468	704	421	633
	34	699	1050	622	935	554	832	497	747	441	663	396	596
	36	660	992	588	883	523	786	470	706	416	626	374	563
	38	625	939	557	837	495	744	445	669	394	593	355	533
	40	594	893	529	795	471	707	423	635	375	563	337	506
	42	566	850	504	757	448	674	403	605	357	536	321	482
	44	540	811	481	723	428	643	384	578	341	512	306	460
	46	516	776	460	691	409	615	368	552	326	490	293	440
	48	495	744	441	663	392	589	352	529	312	469	281	422
	50	475	714	423	636	376	566	338	508	300	451	269	405
	52	457	687	407	612	362	544	325	489	288	433	259	389
	54	440	661	392	589	349	524	313	471	278	417	250	375
	56	424	638	378	568	336	505	302	454	268	402	241	362
	58	410	616	365	548	325	488	291	438	258	388	232	349
	60	396	595	353	530	314	472	282	424	250	375	225	338
	62	383	576	341	513	304	456	273	410	242	363	217	327
	64	371	558	331	497	294	442	264	397	234	352	211	316
	66	360	541	321	482	285	429	256	385	227	341	204	307
	68	349	525	311	468	277	416	249	374	220	331	198	298
	70	339	510	302	454	269	404	242	363	214	322	192	289
	72	330	496	294	442	261	393	235	353	208	313	187	281
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	23800	35700	21200	31800	18800	28300	16900	25400	15000	22500	13500	20300
M_p/Ω_b	$\phi_b M_p$, kip-ft	2970	4460	2640	3980	2350	3540	2110	3180	1870	2820	1680	2530
M_r/Ω_b	$\phi_b M_r$, kip-ft	1820	2730	1620	2440	1450	2180	1310	1960	1160	1750	1050	1580
BF	BF, kips	30.3	45.6	29.8	44.8	29.2	43.9	28.3	42.5	27.0	40.7	25.8	38.7
V_n/Ω_v	$\phi_v V_n$, kips	739	1110	653	980	588	882	520	779	480	719	436	653
Z_x, in.^{3}		1190		1060		943		847		751		675	
L_p, ft		12.7		12.6		12.5		12.4		12.3		12.2	
L_r, ft		50.7		46.9		43.4		40.9		38.7		36.9	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W30

Shape		W30×											
		173		148		132		124		116		108	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	10											650	975
	11					746	1120	706	1060	678	1020	628	944
	12			797	1200	727	1090	679	1020	629	945	576	865
	13			768	1150	671	1010	626	942	580	872	531	798
	14			713	1070	623	936	582	874	539	810	493	741
	15	798	1200	665	1000	582	874	543	816	503	756	460	692
	16	757	1140	624	938	545	819	509	765	472	709	432	649
	17	713	1070	587	882	513	771	479	720	444	667	406	611
	18	673	1010	554	833	485	728	452	680	419	630	384	577
	19	638	958	525	789	459	690	429	644	397	597	363	546
	20	606	911	499	750	436	656	407	612	377	567	345	519
	21	577	867	475	714	415	624	388	583	359	540	329	494
	22	551	828	454	682	396	596	370	556	343	515	314	472
	23	527	792	434	652	379	570	354	532	328	493	300	451
	24	505	759	416	625	363	546	339	510	314	473	288	433
	25	485	728	399	600	349	524	326	490	302	454	276	415
	26	466	700	384	577	335	504	313	471	290	436	266	399
	27	449	674	370	556	323	486	302	453	279	420	256	384
	28	433	650	356	536	312	468	291	437	269	405	247	371
	29	418	628	344	517	301	452	281	422	260	391	238	358
	30	404	607	333	500	291	437	271	408	251	378	230	346
	32	379	569	312	469	273	410	254	383	236	354	216	324
	34	356	536	294	441	257	386	240	360	222	334	203	305
	36	337	506	277	417	242	364	226	340	210	315	192	288
	38	319	479	263	395	230	345	214	322	199	298	182	273
	40	303	455	250	375	218	328	204	306	189	284	173	260
	42	288	434	238	357	208	312	194	291	180	270	164	247
	44	275	414	227	341	198	298	185	278	171	258	157	236
	46	263	396	217	326	190	285	177	266	164	247	150	226
	48	252	379	208	313	182	273	170	255	157	236	144	216
	50	242	364	200	300	174	262	163	245	151	227	138	208
	52	233	350	192	288	168	252	157	235	145	218	133	200
	54	224	337	185	278	162	243	151	227	140	210	128	192
	56	216	325	178	268	156	234	145	219	135	203	123	185
	58	209	314	172	259	150	226	140	211	130	196	119	179
	60	202	303	166	250	145	219	136	204	126	189	115	173
	62	195	294	161	242	141	211	131	197	122	183	111	167
	64	189	285	156	234	136	205	127	191	118	177	108	162
	66	184	276	151	227	132	199	123	185	114	172	105	157
	68	178	268	147	221	128	193	120	180	111	167	102	153
	70	173	260	143	214	125	187	116	175	108	162	98.7	148
	72	168	253	139	208	121	182	113	170	105	158	95.9	144
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	12100	18200	9980	15000	8720	13100	8140	12200	7540	11300	6910	10400
M_p/Ω_b	$\phi_b M_p$, kip-ft	1510	2280	1250	1880	1090	1640	1020	1530	943	1420	863	1300
M_r/Ω_b	$\phi_b M_r$, kip-ft	945	1420	761	1140	664	998	620	932	575	864	522	785
BF	*BF*, kips	24.4	36.6	28.8	43.3	26.9	40.5	25.9	39.0	24.7	37.2	23.7	35.6
V_n/Ω_v	$\phi_v V_n$, kips	399	598	399	598	373	559	353	529	339	509	325	488
Z_x, in.^{3}		607		500		437		408		378		346	
L_p, ft		12.1		8.05		7.95		7.88		7.74		7.59	
L_r, ft		35.5		24.9		23.8		23.2		22.6		22.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W30–W27

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W30×				W27×							
		99		90^v		539^h		368^h		336^h		307^h	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	10	617	925										
	11	566	851	499	749								
	12	519	780	471	708								
	13	479	720	435	653								
	14	445	669	403	606	2560	3840	1680	2520	1510	2270		
	15	415	624	377	566	2510	3780	1650	2480	1500	2260	1370	2060
	16	389	585	353	531	2360	3540	1550	2330	1410	2120	1280	1930
	17	366	551	332	499	2220	3340	1460	2190	1330	1990	1210	1820
	18	346	520	314	472	2100	3150	1380	2070	1250	1880	1140	1720
	19	328	493	297	447	1990	2980	1300	1960	1190	1780	1080	1630
	20	311	468	282	425	1890	2840	1240	1860	1130	1700	1030	1550
	21	297	446	269	404	1800	2700	1180	1770	1070	1610	979	1470
	22	283	425	257	386	1710	2580	1130	1690	1030	1540	934	1400
	23	271	407	246	369	1640	2470	1080	1620	981	1470	894	1340
	24	259	390	235	354	1570	2360	1030	1550	940	1410	857	1290
	25	249	374	226	340	1510	2270	990	1490	902	1360	822	1240
	26	240	360	217	327	1450	2180	952	1430	867	1300	791	1190
	27	231	347	209	314	1400	2100	917	1380	835	1260	761	1140
	28	222	334	202	303	1350	2030	884	1330	806	1210	734	1100
	29	215	323	195	293	1300	1960	853	1280	778	1170	709	1070
	30	208	312	188	283	1260	1890	825	1240	752	1130	685	1030
	32	195	293	177	265	1180	1770	773	1160	705	1060	642	966
	34	183	275	166	250	1110	1670	728	1090	663	997	605	909
	36	173	260	157	236	1050	1580	688	1030	627	942	571	858
	38	164	246	149	223	993	1490	651	979	594	892	541	813
	40	156	234	141	212	943	1420	619	930	564	848	514	773
	42	148	223	134	202	898	1350	589	886	537	807	489	736
	44	142	213	128	193	857	1290	563	845	513	770	467	702
	46	135	203	123	185	820	1230	538	809	490	737	447	672
	48	130	195	118	177	786	1180	516	775	470	706	428	644
	50	125	187	113	170	754	1130	495	744	451	678	411	618
	52	120	180	109	163	725	1090	476	715	434	652	395	594
	54	115	173	105	157	699	1050	458	689	418	628	381	572
	56	111	167	101	152	674	1010	442	664	403	605	367	552
	58	107	161	97.4	146	650	978	427	641	389	584	354	533
	60	104	156	94.1	142	629	945	413	620	376	565	343	515
	62	100	151	91.1	137	608	915	399	600	364	547	332	498
	64	97.3	146	88.3	133	589	886	387	581	352	530	321	483
	66	94.4	142	85.6	129	572	859	375	564	342	514	311	468
	68	91.6	138	83.1	125	555	834	364	547	332	499	302	454
	70	89.0	134	80.7	121	539	810	354	531	322	484	294	441
	72	86.5	130	78.5	118	524	788	344	517	313	471	286	429
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	6230	9360	5650	8490	37700	56700	24800	37200	22600	33900	20600	30900
M_p/Ω_b	$\phi_b M_p$, kip-ft	778	1170	706	1060	4720	7090	3090	4650	2820	4240	2570	3860
M_r/Ω_b	$\phi_b M_r$, kip-ft	470	706	428	643	2740	4120	1850	2780	1700	2550	1550	2330
BF	BF, kips	22.2	33.3	20.5	30.9	26.1	39.2	25.1	37.7	25.1	37.7	25.2	37.8
V_n/Ω_v	$\phi_v V_n$, kips	308	463	249	375	1280	1920	839	1260	756	1130	687	1030
Z_x, in.^{3}		312		283		1890		1240		1130		1030	
L_p, ft		7.42		7.38		12.9		12.3		12.2		12.0	
L_r, ft		21.4		20.9		88.6		61.9		56.9		52.6	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with F_y = 50 ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W27

Shape		W27×											
		281		258		235		217		194		178	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	14			1140		1040	1560			843		806	1210
	15	1240	1860	1130	1700	1030	1540	944	1420	840	1260	758	1140
	16	1170	1760	1060	1600	963	1450	887	1330	787	1180	711	1070
	17	1100	1650	1000	1500	906	1360	835	1250	741	1110	669	1010
	18	1040	1560	945	1420	856	1290	788	1190	700	1050	632	950
	19	983	1480	895	1350	811	1220	747	1120	663	996	599	900
	20	934	1400	850	1280	770	1160	710	1070	630	947	569	855
	21	890	1340	810	1220	734	1100	676	1020	600	901	542	814
	22	849	1280	773	1160	700	1050	645	970	572	860	517	777
	23	812	1220	739	1110	670	1010	617	927	548	823	495	743
	24	778	1170	709	1070	642	965	591	889	525	789	474	713
	25	747	1120	680	1020	616	926	568	853	504	757	455	684
	26	719	1080	654	983	593	891	546	820	484	728	438	658
	27	692	1040	630	947	571	858	526	790	466	701	421	633
	28	667	1000	607	913	550	827	507	762	450	676	406	611
	29	644	968	586	881	531	799	489	736	434	653	392	590
	30	623	936	567	852	514	772	473	711	420	631	379	570
	32	584	878	531	799	482	724	443	667	394	592	356	534
	34	549	826	500	752	453	681	417	627	370	557	335	503
	36	519	780	472	710	428	643	394	593	350	526	316	475
	38	492	739	448	673	406	609	373	561	331	498	299	450
	40	467	702	425	639	385	579	355	533	315	473	284	428
	42	445	669	405	609	367	551	338	508	300	451	271	407
	44	425	638	386	581	350	526	323	485	286	430	259	389
	46	406	610	370	556	335	503	309	464	274	412	247	372
	48	389	585	354	533	321	483	296	444	262	394	237	356
	50	374	562	340	511	308	463	284	427	252	379	228	342
	52	359	540	327	492	296	445	273	410	242	364	219	329
	54	346	520	315	473	285	429	263	395	233	351	211	317
	56	334	501	304	456	275	414	253	381	225	338	203	305
	58	322	484	293	441	266	399	245	368	217	326	196	295
	60	311	468	283	426	257	386	237	356	210	316	190	285
	62	301	453	274	412	249	374	229	344	203	305	184	276
	64	292	439	266	399	241	362	222	333	197	296	178	267
	66	283	425	258	387	233	351	215	323	191	287	172	259
	68	275	413	250	376	227	341	209	314	185	278	167	251
	70	267	401	243	365	220	331	203	305	180	270		
	72	259	390	236	355								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	18700	28100	17000	25600	15400	23200	14200	21300	12600	18900	11400	17100
M_p/Ω_b	$\phi_b M_p$, kip-ft	2340	3510	2130	3200	1930	2900	1770	2670	1570	2370	1420	2140
M_r/Ω_b	$\phi_b M_r$, kip-ft	1420	2140	1300	1960	1180	1780	1100	1650	976	1470	882	1330
BF	BF, kips	24.6	36.9	24.2	36.4	23.9	35.9	23.3	35.1	22.5	33.8	21.7	32.7
V_n/Ω_v	$\phi_v V_n$, kips	621	931	568	852	522	782	472	708	422	632	403	605
Z_x, in.^{3}		936		852		772		711		631		570	
L_p, ft		12.0		11.9		11.8		11.7		11.6		11.5	
L_r, ft		49.2		45.9		42.9		40.8		38.2		36.3	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W27

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W27×											
		161		146		129		114		102		94	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	10									558	837	528	791
	11					674	1010	622	933	553	832	504	758
	12					657	988	571	858	507	763	462	695
	13			663		606	912	527	792	468	704	427	642
	14	728	1090	662	994	563	846	489	735	435	654	396	596
	15	685	1030	617	928	526	790	456	686	406	610	370	556
	16	642	966	579	870	493	741	428	643	380	572	347	521
	17	605	909	545	819	464	697	403	605	358	538	326	491
	18	571	858	515	773	438	658	380	572	338	508	308	463
	19	541	813	487	733	415	624	360	542	320	482	292	439
	20	514	773	463	696	394	593	342	515	304	458	277	417
	21	489	736	441	663	375	564	326	490	290	436	264	397
	22	467	702	421	633	358	539	311	468	277	416	252	379
	23	447	672	403	605	343	515	298	447	265	398	241	363
	24	428	644	386	580	329	494	285	429	254	381	231	348
	25	411	618	370	557	315	474	274	412	244	366	222	334
	26	395	594	356	535	303	456	263	396	234	352	213	321
	27	381	572	343	516	292	439	254	381	225	339	206	309
	28	367	552	331	497	282	423	245	368	217	327	198	298
	29	354	533	319	480	272	409	236	355	210	316	191	288
	30	343	515	309	464	263	395	228	343	203	305	185	278
	32	321	483	289	435	246	370	214	322	190	286	173	261
	34	302	454	272	409	232	349	201	303	179	269	163	245
	36	286	429	257	387	219	329	190	286	169	254	154	232
	38	271	407	244	366	207	312	180	271	160	241	146	219
	40	257	386	232	348	197	296	171	257	152	229	139	209
	42	245	368	221	331	188	282	163	245	145	218	132	199
	44	234	351	210	316	179	269	156	234	138	208	126	190
	46	223	336	201	303	171	258	149	224	132	199	121	181
	48	214	322	193	290	164	247	143	214	127	191	116	174
	50	206	309	185	278	158	237	137	206	122	183	111	167
	52	198	297	178	268	152	228	132	198	117	176	107	160
	54	190	286	172	258	146	219	127	191	113	169	103	154
	56	184	276	165	249	141	212	122	184	109	163	99.1	149
	58	177	266	160	240	136	204	118	177	105	158	95.7	144
	60	171	258	154	232	131	198	114	172	101	153	92.5	139
	62	166	249	149	225	127	191	110	166	98.2	148	89.5	135
	64	161	241	145	218	123	185	107	161	95.1	143	86.7	130
	66	156	234	140	211	119	180	104	156	92.2	139	84.1	126
	68	151	227	136	205	116	174	101	151				

Beam Properties

		161 ASD	161 LRFD	146 ASD	146 LRFD	129 ASD	129 LRFD	114 ASD	114 LRFD	102 ASD	102 LRFD	94 ASD	94 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	10300	15500	9260	13900	7880	11900	6850	10300	6090	9150	5550	8340
M_p/Ω_b	$\phi_b M_p$, kip-ft	1280	1930	1160	1740	986	1480	856	1290	761	1140	694	1040
M_r/Ω_b	$\phi_b M_r$, kip-ft	800	1200	723	1090	603	906	522	785	466	701	424	638
BF	BF, kips	20.8	31.3	19.7	29.6	23.3	35.0	21.7	32.6	20.2	30.3	19.1	28.8
V_n/Ω_v	$\phi_v V_n$, kips	364	546	331	497	337	506	311	467	279	419	264	396

	161	146	129	114	102	94
Z_x, in.^{3}	515	464	395	343	305	278
L_p, ft	11.4	11.3	7.81	7.70	7.59	7.49
L_r, ft	34.7	33.4	24.3	23.1	22.2	21.6

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W27–W24

Shape		W27×		W24×									
		84		370^h		335^h		306^h		279^h		250	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	9	491	737										
	10	487	732										
	11	443	665										
	12	406	610										
	13	375	563	1700	2550	1520	2280	1370	2050	1240	1860	1100	1640
	14	348	523	1610	2420	1450	2190	1310	1980	1190	1790	1060	1590
	15	325	488	1500	2260	1360	2040	1230	1840	1110	1670	990	1490
	16	304	458	1410	2120	1270	1910	1150	1730	1040	1570	928	1400
	17	286	431	1330	1990	1200	1800	1080	1630	980	1470	874	1310
	18	271	407	1250	1880	1130	1700	1020	1540	926	1390	825	1240
	19	256	385	1190	1780	1070	1610	969	1460	877	1320	782	1170
	20	244	366	1130	1700	1020	1530	920	1380	833	1250	743	1120
	21	232	349	1070	1610	969	1460	876	1320	794	1190	707	1060
	22	221	333	1030	1540	925	1390	837	1260	758	1140	675	1010
	23	212	318	981	1470	885	1330	800	1200	725	1090	646	970
	24	203	305	940	1410	848	1280	767	1150	694	1040	619	930
	25	195	293	902	1360	814	1220	736	1110	667	1000	594	893
	26	187	282	867	1300	783	1180	708	1060	641	963	571	858
	27	180	271	835	1260	754	1130	682	1020	617	928	550	827
	28	174	261	806	1210	727	1090	657	988	595	895	530	797
	29	168	252	778	1170	702	1060	635	954	575	864	512	770
	30	162	244	752	1130	679	1020	613	922	556	835	495	744
	32	152	229	705	1060	636	956	575	864	521	783	464	698
	34	143	215	663	997	599	900	541	814	490	737	437	656
	36	135	203	627	942	566	850	511	768	463	696	413	620
	38	128	193	594	892	536	805	484	728	439	659	391	587
	40	122	183	564	848	509	765	460	692	417	626	371	558
	42	116	174	537	807	485	729	438	659	397	596	354	531
	44	111	166	513	770	463	695	418	629	379	569	338	507
	46	106	159	490	737	443	665	400	601	362	545	323	485
	48	101	153	470	706	424	638	383	576	347	522	309	465
	50	97.4	146	451	678	407	612	368	553	333	501	297	446
	52	93.7	141	434	652	392	588	354	532	321	482	286	429
	54	90.2	136	418	628	377	567	341	512	309	464	275	413
	56	87.0	131	403	605	364	546	329	494	298	447	265	399
	58	84.0	126	389	584	351	528	317	477	287	432	256	385
	60	81.2	122	376	565	339	510	307	461	278	418	248	372
	62	78.6	118	364	547	328	494	297	446	269	404	240	360
	64	76.1	114	352	530	318	478	288	432	260	391	232	349
	66	73.8	111	342	514	308	464	279	419	253	380		
	68			332	499	299	450						
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	4870	7320	22600	33900	20400	30600	18400	27700	16700	25100	14900	22300
M_p/Ω_b	$\phi_b M_p$, kip-ft	609	915	2820	4240	2540	3830	2300	3460	2080	3130	1860	2790
M_r/Ω_b	$\phi_b M_r$, kip-ft	372	559	1670	2510	1510	2270	1380	2070	1250	1880	1120	1690
BF	*BF*, kips	17.6	26.4	19.9	29.9	20.1	30.2	19.9	29.8	19.7	29.6	19.5	29.3
V_n/Ω_v	$\phi_v V_n$, kips	246	369	851	1280	760	1140	684	1030	620	930	548	822
Z_x, in.^{3}		244		1130		1020		922		835		744	
L_p, ft		7.31		11.6		11.4		11.3		11.2		11.1	
L_r, ft		20.8		69.2		63.0		57.8		53.4		48.6	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W24

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape	W24×											
	229		207		192		176		162		146	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 12											643	
13	999	1500	895	1340	825	1240	757	1140	705	1060	642	965
14	962	1450	864	1300	797	1200	729	1100	667	1000	596	896
15	898	1350	806	1210	744	1120	680	1020	623	936	556	836
16	842	1270	756	1140	697	1050	637	958	584	878	521	784
17	793	1190	712	1070	656	986	600	902	549	826	491	738
18	749	1130	672	1010	620	932	567	852	519	780	464	697
19	709	1070	637	957	587	883	537	807	492	739	439	660
20	674	1010	605	909	558	839	510	767	467	702	417	627
21	642	964	576	866	531	799	486	730	445	669	397	597
22	612	920	550	826	507	762	464	697	425	638	379	570
23	586	880	526	790	485	729	443	667	406	610	363	545
24	561	844	504	758	465	699	425	639	389	585	348	523
25	539	810	484	727	446	671	408	613	374	562	334	502
26	518	779	465	699	429	645	392	590	359	540	321	482
27	499	750	448	673	413	621	378	568	346	520	309	464
28	481	723	432	649	398	599	364	548	334	501	298	448
29	465	698	417	627	385	578	352	529	322	484	288	432
30	449	675	403	606	372	559	340	511	311	468	278	418
32	421	633	378	568	349	524	319	479	292	439	261	392
34	396	596	356	535	328	493	300	451	275	413	245	369
36	374	563	336	505	310	466	283	426	259	390	232	348
38	355	533	318	478	294	441	268	403	246	369	220	330
40	337	506	302	455	279	419	255	383	234	351	209	314
42	321	482	288	433	266	399	243	365	222	334	199	299
44	306	460	275	413	254	381	232	348	212	319	190	285
46	293	440	263	395	243	365	222	333	203	305	181	273
48	281	422	252	379	232	349	212	319	195	293	174	261
50	269	405	242	364	223	335	204	307	187	281	167	251
52	259	389	233	350	215	323	196	295	180	270	160	241
54	250	375	224	337	207	311	189	284	173	260	155	232
56	241	362	216	325	199	299	182	274	167	251	149	224
58	232	349	209	313	192	289	176	264	161	242	144	216
60	225	338	202	303	186	280	170	255	156	234	139	209
62	217	327	195	293	180	270	165	247	151	226		
64	211	316	189	284								

Beam Properties

ASD	LRFD	229 ASD	229 LRFD	207 ASD	207 LRFD	192 ASD	192 LRFD	176 ASD	176 LRFD	162 ASD	162 LRFD	146 ASD	146 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	13500	20300	12100	18200	11200	16800	10200	15300	9340	14000	8340	12500
M_p/Ω_b	$\phi_b M_p$, kip-ft	1680	2530	1510	2270	1390	2100	1270	1920	1170	1760	1040	1570
M_r/Ω_b	$\phi_b M_r$, kip-ft	1030	1540	927	1390	858	1290	786	1180	723	1090	648	974
BF	BF, kips	19.2	28.9	18.9	28.5	18.7	28.0	18.3	27.6	17.8	26.8	17.1	25.8
V_n/Ω_v	$\phi_v V_n$, kips	500	749	447	671	413	619	379	568	353	529	322	482
Z_x, in.^{3}		675		606		559		511		468		418	
L_p, ft		11.0		10.9		10.8		10.7		10.8		10.6	
L_r, ft		45.2		41.8		39.6		37.4		35.7		33.7	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W24

Shape		W24×											
		131		117		104		103		94		84	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	9											453	680
	10							540	809	501	751	447	672
	11							508	764	461	693	406	611
	12	592	889	534	801	481	722	466	700	422	635	373	560
	13	568	854	502	755	444	667	430	646	390	586	344	517
	14	528	793	466	701	412	619	399	600	362	544	319	480
	15	492	740	435	654	385	578	373	560	338	508	298	448
	16	462	694	408	613	361	542	349	525	317	476	279	420
	17	434	653	384	577	339	510	329	494	298	448	263	395
	18	410	617	363	545	320	482	310	467	282	423	248	373
	19	389	584	344	516	304	456	294	442	267	401	235	354
	20	369	555	326	491	288	434	279	420	253	381	224	336
	21	352	529	311	467	275	413	266	400	241	363	213	320
	22	336	505	297	446	262	394	254	382	230	346	203	305
	23	321	483	284	427	251	377	243	365	220	331	194	292
	24	308	463	272	409	240	361	233	350	211	317	186	280
	25	295	444	261	392	231	347	224	336	203	305	179	269
	26	284	427	251	377	222	333	215	323	195	293	172	258
	27	274	411	242	363	214	321	207	311	188	282	166	249
	28	264	396	233	350	206	310	200	300	181	272	160	240
	29	255	383	225	338	199	299	193	290	175	263	154	232
	30	246	370	218	327	192	289	186	280	169	254	149	224
	32	231	347	204	307	180	271	175	263	158	238	140	210
	34	217	326	192	289	170	255	164	247	149	224	132	198
	36	205	308	181	273	160	241	155	233	141	212	124	187
	38	194	292	172	258	152	228	147	221	133	201	118	177
	40	185	278	163	245	144	217	140	210	127	191	112	168
	42	176	264	155	234	137	206	133	200	121	181	106	160
	44	168	252	148	223	131	197	127	191	115	173	102	153
	46	161	241	142	213	125	188	121	183	110	166	97.2	146
	48	154	231	136	204	120	181	116	175	106	159	93.1	140
	50	148	222	131	196	115	173	112	168	101	152	89.4	134
	52	142	213	126	189	111	167	107	162	97.5	147	86.0	129
	54	137	206	121	182	107	161	103	156	93.9	141	82.8	124
	56	132	198	117	175	103	155	99.8	150	90.5	136	79.8	120
	58	127	191	113	169	99.5	149	96.4	145	87.4	131	77.1	116
	60	123	185	109	164	96.1	145	93.1	140	84.5	127	74.5	112
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	7390	11100	6530	9810	5770	8670	5590	8400	5070	7620	4470	6720
M_p/Ω_b	$\phi_b M_p$, kip-ft	923	1390	816	1230	721	1080	699	1050	634	953	559	840
M_r/Ω_b	$\phi_b M_r$, kip-ft	575	864	508	764	451	677	428	643	388	583	342	515
BF	BF, kips	16.3	24.5	15.3	23.1	14.3	21.5	18.2	27.4	17.3	26.0	16.2	24.3
V_n/Ω_v	$\phi_v V_n$, kips	296	444	267	400	241	361	270	405	250	376	227	340
Z_x, in.^{3}		370		327		289		280		254		224	
L_p, ft		10.5		10.4		10.3		7.03		6.99		6.89	
L_r, ft		31.9		30.4		29.2		21.9		21.2		20.3	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W24–W21

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W24×								W21×			
		76		68		62		55^v		201		182	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	7					408	612						
	8			394	591	382	574	334	503				
	9	421	631	393	590	339	510	297	447				
	10	399	600	353	531	305	459	267	402				
	11	363	545	321	483	278	417	243	365				
	12	333	500	294	443	254	383	223	335	838	1260	754	1130
	13	307	462	272	408	235	353	206	309	814	1220	731	1100
	14	285	429	252	379	218	328	191	287	756	1140	679	1020
	15	266	400	236	354	204	306	178	268	705	1060	633	952
	16	250	375	221	332	191	287	167	251	661	994	594	893
	17	235	353	208	312	180	270	157	236	622	935	559	840
	18	222	333	196	295	170	255	149	223	588	883	528	793
	19	210	316	186	279	161	242	141	212	557	837	500	752
	20	200	300	177	266	153	230	134	201	529	795	475	714
	21	190	286	168	253	145	219	127	191	504	757	452	680
	22	181	273	161	241	139	209	122	183	481	723	432	649
	23	174	261	154	231	133	200	116	175	460	691	413	621
	24	166	250	147	221	127	191	111	168	441	663	396	595
	25	160	240	141	212	122	184	107	161	423	636	380	571
	26	154	231	136	204	117	177	103	155	407	612	365	549
	27	148	222	131	197	113	170	99.1	149	392	589	352	529
	28	143	214	126	190	109	164	95.5	144	378	568	339	510
	29	138	207	122	183	105	158	92.2	139	365	548	328	492
	30	133	200	118	177	102	153	89.2	134	353	530	317	476
	32	125	188	110	166	95.4	143	83.6	126	331	497	297	446
	34	117	176	104	156	89.8	135	78.7	118	311	468	279	420
	36	111	167	98.1	148	84.8	128	74.3	112	294	442	264	397
	38	105	158	93.0	140	80.4	121	70.4	106	278	418	250	376
	40	99.8	150	88.3	133	76.3	115	66.9	101	264	398	238	357
	42	95.0	143	84.1	126	72.7	109	63.7	95.7	252	379	226	340
	44	90.7	136	80.3	121	69.4	104	60.8	91.4	240	361	216	325
	46	86.8	130	76.8	115	66.4	99.8	58.1	87.4	230	346	207	310
	48	83.2	125	73.6	111	63.6	95.6	55.7	83.8	220	331	198	298
	50	79.8	120	70.7	106	61.1	91.8	53.5	80.4	212	318	190	286
	52	76.8	115	67.9	102	58.7	88.3	51.4	77.3	203	306	183	275
	54	73.9	111	65.4	98.3	56.6	85.0	49.5	74.4	196	294	176	264
	56	71.3	107	63.1	94.8	54.5	82.0	47.8	71.8	189	284	170	255
	58	68.8	103	60.9	91.6	52.7	79.1	46.1	69.3				
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	3990	6000	3530	5310	3050	4590	2670	4020	10600	15900	9500	14300
M_p/Ω_b	$\phi_b M_p$, kip-ft	499	750	442	664	382	574	334	503	1320	1990	1190	1790
M_r/Ω_b	$\phi_b M_r$, kip-ft	307	462	269	404	229	344	199	299	805	1210	728	1090
BF	BF, kips	15.0	22.5	14.1	21.2	16.0	24.1	14.8	22.2	14.6	21.9	14.3	21.6
V_n/Ω_v	$\phi_v V_n$, kips	210	316	197	295	204	306	167	251	419	629	377	566
Z_x, in.^{3}		200		177		153		134		530		476	
L_p, ft		6.78		6.61		4.87		4.73		10.7		10.6	
L_r, ft		19.6		18.8		14.4		13.9		46.1		42.6	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

^{v} Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

F_y = 50 ksi

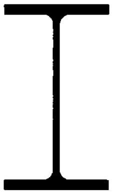
Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W21

Shape		W21×											
		166		147		132		122		111		101	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	11			635	953	568	851	520	780	473	710	427	641
	12	674	1010	620	933	554	833	511	768	464	698	421	633
	13	663	997	573	861	511	768	471	708	428	644	388	584
	14	616	926	532	799	475	714	438	658	398	598	361	542
	15	575	864	496	746	443	666	409	614	371	558	337	506
	16	539	810	465	699	415	624	383	576	348	523	316	474
	17	507	762	438	658	391	588	360	542	328	492	297	446
	18	479	720	414	622	369	555	340	512	309	465	281	422
	19	454	682	392	589	350	526	323	485	293	441	266	399
	20	431	648	372	560	332	500	306	461	278	419	252	380
	21	411	617	355	533	317	476	292	439	265	399	240	361
	22	392	589	338	509	302	454	279	419	253	380	230	345
	23	375	563	324	487	289	434	266	400	242	364	220	330
	24	359	540	310	466	277	416	255	384	232	349	210	316
	25	345	518	298	448	266	400	245	368	223	335	202	304
	26	332	498	286	430	256	384	236	354	214	322	194	292
	27	319	480	276	414	246	370	227	341	206	310	187	281
	28	308	463	266	400	237	357	219	329	199	299	180	271
	29	297	447	257	386	229	344	211	318	192	289	174	262
	30	287	432	248	373	222	333	204	307	186	279	168	253
	32	269	405	233	350	208	312	191	288	174	262	158	237
	34	254	381	219	329	195	294	180	271	164	246	149	223
	36	240	360	207	311	185	278	170	256	155	233	140	211
	38	227	341	196	294	175	263	161	242	147	220	133	200
	40	216	324	186	280	166	250	153	230	139	209	126	190
	42	205	309	177	266	158	238	146	219	133	199	120	181
	44	196	295	169	254	151	227	139	209	127	190	115	173
	46	187	282	162	243	144	217	133	200	121	182	110	165
	48	180	270	155	233	138	208	128	192	116	174	105	158
	50	172	259	149	224	133	200	123	184	111	167	101	152
	52	166	249	143	215	128	192	118	177	107	161	97.1	146
	54	160	240	138	207	123	185	113	171				
	56	154	231										
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	8620	13000	7450	11200	6650	9990	6130	9210	5570	8370	5050	7590
M_p/Ω_b	$\phi_b M_p$, kip-ft	1080	1620	931	1400	831	1250	766	1150	696	1050	631	949
M_r/Ω_b	$\phi_b M_r$, kip-ft	664	998	575	864	515	774	477	717	435	654	396	596
BF	BF, kips	14.2	21.3	13.8	20.7	13.3	20.0	12.9	19.4	12.4	18.7	11.8	17.7
V_n/Ω_v	$\phi_v V_n$, kips	337	506	318	476	284	426	260	390	237	355	214	320
Z_x, in.^{3}		432		373		333		307		279		253	
L_p, ft		10.6		10.4		10.3		10.3		10.2		10.2	
L_r, ft		39.8		36.3		34.1		32.7		31.3		30.1	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.



W21

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W21×									
		93		83		73		68		62	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Span, ft	**8**	502	752	441	662	387	580	363	545	336	504
	9	490	737	435	653	381	573	355	533	319	480
	10	441	663	391	588	343	516	319	480	287	432
	11	401	603	356	535	312	469	290	436	261	393
	12	368	553	326	490	286	430	266	400	240	360
	13	339	510	301	452	264	397	246	369	221	332
	14	315	474	279	420	245	369	228	343	205	309
	15	294	442	261	392	229	344	213	320	192	288
	16	276	414	245	368	215	323	200	300	180	270
	17	259	390	230	346	202	304	188	282	169	254
	18	245	368	217	327	191	287	177	267	160	240
	19	232	349	206	309	181	272	168	253	151	227
	20	221	332	196	294	172	258	160	240	144	216
	21	210	316	186	280	163	246	152	229	137	206
	22	201	301	178	267	156	235	145	218	131	196
	23	192	288	170	256	149	224	139	209	125	188
	24	184	276	163	245	143	215	133	200	120	180
	25	176	265	156	235	137	206	128	192	115	173
	26	170	255	150	226	132	198	123	185	111	166
	27	163	246	145	218	127	191	118	178	106	160
	28	158	237	140	210	123	184	114	171	103	154
	29	152	229	135	203	118	178	110	166	99.1	149
	30	147	221	130	196	114	172	106	160	95.8	144
	32	138	207	122	184	107	161	99.8	150	89.8	135
	34	130	195	115	173	101	152	93.9	141	84.5	127
	36	123	184	109	163	95.4	143	88.7	133	79.8	120
	38	116	174	103	155	90.3	136	84.0	126	75.6	114
	40	110	166	97.8	147	85.8	129	79.8	120	71.9	108
	42	105	158	93.1	140	81.7	123	76.0	114	68.4	103
	44	100	151	88.9	134	78.0	117	72.6	109	65.3	98.2
	46	95.9	144	85.0	128	74.6	112	69.4	104	62.5	93.9
	48	91.9	138	81.5	122	71.5	107	66.5	100	59.9	90.0
	50	88.2	133	78.2	118	68.7	103	63.9	96.0	57.5	86.4
	52	84.8	128	75.2	113	66.0	99.2	61.4	92.3	55.3	83.1
	54	81.7	123								
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	4410	6630	3910	5880	3430	5160	3190	4800	2870	4320
M_p/Ω_b	$\phi_b M_p$, kip-ft	551	829	489	735	429	645	399	600	359	540
M_r/Ω_b	$\phi_b M_r$, kip-ft	335	504	299	449	264	396	245	368	222	333
BF	BF, kips	14.6	21.9	13.8	20.8	12.9	19.4	12.5	18.8	11.6	17.4
V_n/Ω_v	$\phi_v V_n$, kips	251	376	221	331	193	290	182	273	168	252
Z_x, in.^{3}		221		196		172		160		144	
L_p, ft		6.50		6.46		6.39		6.36		6.25	
L_r, ft		21.3		20.2		19.2		18.7		18.1	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W21

Shape		W21×									
		57		55		50		48^f		44	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	6					317	475			289	434
	7	341	512			314	471	289	433	272	409
	8	322	484	312	468	274	413	265	398	238	358
	9	286	430	279	420	244	367	235	354	212	318
	10	257	387	251	378	220	330	212	318	190	286
	11	234	352	229	344	200	300	193	289	173	260
	12	215	323	210	315	183	275	177	265	159	239
	13	198	298	193	291	169	254	163	245	146	220
	14	184	276	180	270	157	236	151	227	136	204
	15	172	258	168	252	146	220	141	212	127	191
	16	161	242	157	236	137	206	132	199	119	179
	17	151	228	148	222	129	194	125	187	112	168
	18	143	215	140	210	122	183	118	177	106	159
	19	136	204	132	199	116	174	111	168	100	151
	20	129	194	126	189	110	165	106	159	95.2	143
	21	123	184	120	180	105	157	101	152	90.7	136
	22	117	176	114	172	99.8	150	96.3	145	86.6	130
	23	112	168	109	164	95.5	143	92.1	138	82.8	124
	24	107	161	105	158	91.5	138	88.3	133	79.3	119
	25	103	155	101	151	87.8	132	84.7	127	76.2	114
	26	99.0	149	96.7	145	84.4	127	81.5	122	73.2	110
	27	95.4	143	93.1	140	81.3	122	78.5	118	70.5	106
	28	92.0	138	89.8	135	78.4	118	75.6	114	68.0	102
	29	88.8	133	86.7	130	75.7	114	73.0	110	65.7	98.7
	30	85.8	129	83.8	126	73.2	110	70.6	106	63.5	95.4
	32	80.5	121	78.6	118	68.6	103	66.2	99.5	59.5	89.4
	34	75.7	114	74.0	111	64.6	97.1	62.3	93.6	56.0	84.2
	36	71.5	108	69.9	105	61.0	91.7	58.8	88.4	52.9	79.5
	38	67.8	102	66.2	99.5	57.8	86.8	55.7	83.8	50.1	75.3
	40	64.4	96.8	62.9	94.5	54.9	82.5	53.0	79.6	47.6	71.6
	42	61.3	92.1	59.9	90.0	52.3	78.6	50.4	75.8	45.3	68.1
	44	58.5	88.0	57.2	85.9	49.9	75.0	48.1	72.4	43.3	65.0
	46	56.0	84.1	54.7	82.2	47.7	71.7	46.0	69.2	41.4	62.2
	48	53.6	80.6	52.4	78.8	45.7	68.8	44.1	66.3	39.7	59.6
	50	51.5	77.4	50.3	75.6	43.9	66.0	42.4	63.7	38.1	57.2
	52	49.5	74.4	48.4	72.7	42.2	63.5				
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	2570	3870	2510	3780	2200	3300	2120	3180	1900	2860
M_p/Ω_b	$\phi_b M_p$, kip-ft	322	484	314	473	274	413	265	398	238	358
M_r/Ω_b	$\phi_b M_r$, kip-ft	194	291	192	289	165	248	162	244	143	214
BF	BF, kips	13.4	20.1	10.8	16.3	12.2	18.3	9.78	14.7	11.2	16.8
V_n/Ω_v	$\phi_v V_n$, kips	171	256	156	234	158	237	144	217	145	217
Z_x, in.^{3}		129		126		110		107		95.4	
L_p, ft		4.77		6.11		4.59		6.09		4.45	
L_r, ft		14.3		17.4		13.6		16.6		13.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W18

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W18×											
		311^h		283^h		258^h		234^h		211		192	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	11	1360	2040	1220	1840	1100	1650	977	1470	876	1310	781	1170
	12	1250	1890	1120	1690	1020	1530	913	1370	815	1230	735	1110
	13	1160	1740	1040	1560	938	1410	843	1270	752	1130	679	1020
	14	1070	1620	964	1450	871	1310	783	1180	699	1050	630	947
	15	1000	1510	900	1350	813	1220	731	1100	652	980	588	884
	16	941	1410	843	1270	762	1150	685	1030	611	919	551	829
	17	885	1330	794	1190	717	1080	645	969	575	865	519	780
	18	836	1260	750	1130	678	1020	609	915	543	817	490	737
	19	792	1190	710	1070	642	965	577	867	515	774	464	698
	20	752	1130	675	1010	610	917	548	824	489	735	441	663
	21	717	1080	643	966	581	873	522	784	466	700	420	631
	22	684	1030	613	922	554	833	498	749	445	668	401	603
	23	654	983	587	882	530	797	476	716	425	639	384	577
	24	627	943	562	845	508	764	457	686	408	613	368	553
	25	602	905	540	811	488	733	438	659	391	588	353	530
	26	579	870	519	780	469	705	421	633	376	565	339	510
	27	557	838	500	751	452	679	406	610	362	544	327	491
	28	537	808	482	724	436	655	391	588	349	525	315	474
	29	519	780	465	699	421	632	378	568	337	507	304	457
	30	502	754	450	676	407	611	365	549	326	490	294	442
	31	485	730	435	654	393	591	353	531	315	474	285	428
	32	470	707	422	634	381	573	342	515	306	459	276	414
	33	456	685	409	615	370	555	332	499	296	445	267	402
	34	443	665	397	596	359	539	322	484	288	432	259	390
	35	430	646	386	579	348	524	313	471	279	420	252	379
	36	418	628	375	563	339	509	304	458	272	408	245	368
	37	407	611	365	548	330	495	296	445	264	397	238	358
	38	396	595	355	534	321	482	288	433	257	387	232	349
	39	386	580	346	520	313	470	281	422	251	377	226	340
	40	376	566	337	507	305	458	274	412	245	368	221	332
	42	358	539	321	483	290	436	261	392	233	350	210	316
	44	342	514	307	461	277	417	249	374	222	334	201	301
	46	327	492	293	441	265	398	238	358	213	320	192	288
	48	314	471	281	422	254	382	228	343	204	306	184	276
	50	301	452	270	406	244	367	219	329	196	294	176	265

Beam Properties

W_c/Ω_b	$\phi_b W_c$, kip-ft	15000	22600	13500	20300	12200	18300	11000	16500	9780	14700	8820	13300
M_p/Ω_b	$\phi_b M_p$, kip-ft	1880	2830	1690	2540	1520	2290	1370	2060	1220	1840	1100	1660
M_r/Ω_b	$\phi_b M_r$, kip-ft	1090	1640	987	1480	898	1350	814	1220	732	1100	664	998
BF	BF, kips	11.2	16.8	11.0	16.6	11.0	16.5	10.8	16.2	10.7	16.1	10.7	16.0
V_n/Ω_v	$\phi_v V_n$, kips	679	1020	612	918	549	824	489	733	438	657	391	586
Z_x, in.^{3}		754		676		611		549		490		442	
L_p, ft		10.4		10.3		10.2		10.1		9.96		9.85	
L_r, ft		81.2		73.8		67.4		61.5		55.8		51.1	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W18

Shape	W18×											
	175		158		143		130		119		106	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 10									497	746	442	663
11	713	1070	639	958	569	854	516	774	475	715	417	627
12	662	995	592	890	536	805	482	725	436	655	383	575
13	611	918	547	822	494	743	445	669	402	605	353	531
14	567	853	508	763	459	690	413	621	374	561	328	493
15	530	796	474	712	428	644	386	580	349	524	306	460
16	497	746	444	668	402	604	362	544	327	491	287	431
17	467	702	418	628	378	568	340	512	308	462	270	406
18	441	663	395	593	357	537	322	483	291	437	255	383
19	418	628	374	562	338	508	305	458	275	414	242	363
20	397	597	355	534	321	483	289	435	261	393	230	345
21	378	569	338	509	306	460	276	414	249	374	219	329
22	361	543	323	485	292	439	263	395	238	357	209	314
23	345	519	309	464	279	420	252	378	227	342	200	300
24	331	497	296	445	268	403	241	362	218	328	191	288
25	318	478	284	427	257	386	232	348	209	314	184	276
26	306	459	273	411	247	372	223	335	201	302	177	265
27	294	442	263	396	238	358	214	322	194	291	170	256
28	284	426	254	381	230	345	207	311	187	281	164	246
29	274	412	245	368	222	333	200	300	180	271	158	238
30	265	398	237	356	214	322	193	290	174	262	153	230
31	256	385	229	345	207	312	187	281	169	254	148	223
32	248	373	222	334	201	302	181	272	163	246	143	216
33	241	362	215	324	195	293	175	264	158	238	139	209
34	234	351	209	314	189	284	170	256	154	231	135	203
35	227	341	203	305	184	276	165	249	149	225	131	197
36	221	332	197	297	179	268	161	242	145	218	128	192
37	215	323	192	289	174	261	156	235	141	212	124	186
38	209	314	187	281	169	254	152	229	138	207	121	182
39	204	306	182	274	165	248	148	223	134	202	118	177
40	199	299	178	267	161	242	145	218	131	197	115	173
42	189	284	169	254	153	230	138	207	125	187	109	164
44	181	271	161	243	146	220	132	198	119	179	104	157
46	173	260	154	232	140	210	126	189	114	171	99.8	150
48	166	249	148	222	134	201	121	181				
50	159	239										

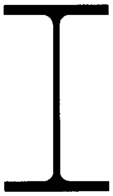
Beam Properties

		175 ASD	175 LRFD	158 ASD	158 LRFD	143 ASD	143 LRFD	130 ASD	130 LRFD	119 ASD	119 LRFD	106 ASD	106 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	7940	11900	7110	10700	6430	9660	5790	8700	5230	7860	4590	6900
M_p/Ω_b	$\phi_b M_p$, kip-ft	993	1490	888	1340	803	1210	724	1090	654	983	574	863
M_r/Ω_b	$\phi_b M_r$, kip-ft	601	903	541	814	493	740	447	672	403	606	356	536
BF	BF, kips	10.6	15.9	10.5	15.7	10.4	15.6	10.2	15.3	10.1	15.2	9.70	14.6
V_n/Ω_v	$\phi_v V_n$, kips	357	535	319	479	285	427	258	387	249	373	221	332

	175	158	143	130	119	106
Z_x, in.^{3}	398	356	322	290	262	230
L_p, ft	9.75	9.68	9.61	9.54	9.50	9.40
L_r, ft	46.7	42.8	39.6	36.7	34.3	31.8

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.



W18

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

$F_y = 50$ ksi

Shape		W18×											
		97		86		76		71		65		60	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	7							366	549				
	8							364	548	330	495	303	454
	9							324	487	295	443	273	410
	10	398	597	353	530	310	464	291	438	265	399	246	369
	11	383	575	338	507	296	445	265	398	241	363	223	335
	12	351	528	309	465	271	407	243	365	221	332	205	308
	13	324	487	286	429	250	376	224	337	204	307	189	284
	14	301	452	265	399	232	349	208	313	190	285	175	264
	15	281	422	248	372	217	326	194	292	177	266	164	246
	16	263	396	232	349	203	306	182	274	166	249	153	231
	17	248	372	218	328	191	288	171	258	156	235	144	217
	18	234	352	206	310	181	272	162	243	147	222	136	205
	19	222	333	195	294	171	257	153	231	140	210	129	194
	20	211	317	186	279	163	245	146	219	133	200	123	185
	21	201	301	177	266	155	233	139	209	126	190	117	176
	22	191	288	169	254	148	222	132	199	121	181	112	168
	23	183	275	161	243	141	213	127	190	115	173	107	160
	24	175	264	155	233	136	204	121	183	111	166	102	154
	25	168	253	149	223	130	196	117	175	106	160	98.2	148
	26	162	243	143	215	125	188	112	168	102	153	94.4	142
	27	156	234	138	207	120	181	108	162	98.3	148	90.9	137
	28	150	226	133	199	116	175	104	156	94.8	143	87.7	132
	29	145	218	128	192	112	169	100	151	91.5	138	84.7	127
	30	140	211	124	186	108	163	97.1	146	88.5	133	81.8	123
	31	136	204	120	180	105	158	94.0	141	85.6	129	79.2	119
	32	132	198	116	174	102	153	91.1	137	83.0	125	76.7	115
	33	128	192	113	169	98.6	148	88.3	133	80.4	121	74.4	112
	34	124	186	109	164	95.7	144	85.7	129	78.1	117	72.2	109
	35	120	181	106	159	93.0	140	83.3	125	75.8	114	70.1	105
	36	117	176	103	155	90.4	136	80.9	122	73.7	111	68.2	103
	37	114	171	100	151	87.9	132	78.8	118	71.7	108	66.4	99.7
	38	111	167	97.7	147	85.6	129	76.7	115	69.9	105	64.6	97.1
	39	108	162	95.2	143	83.4	125	74.7	112	68.1	102	63.0	94.6
	40	105	158	92.8	140	81.3	122	72.9	110	66.4	99.8	61.4	92.3
	42	100	151	88.4	133	77.5	116	69.4	104	63.2	95.0	58.5	87.9
	44	95.7	144	84.4	127	73.9	111	66.2	99.5	60.3	90.7	55.8	83.9
	46	91.6	138					63.4	95.2				
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	4210	6330	3710	5580	3250	4890	2910	4380	2650	3990	2460	3690
M_p/Ω_b	$\phi_b M_p$, kip-ft	526	791	464	698	407	611	364	548	332	499	307	461
M_r/Ω_b	$\phi_b M_r$, kip-ft	328	494	290	436	255	383	222	333	204	307	189	284
BF	BF, kips	9.45	14.2	9.04	13.6	8.49	12.8	10.5	15.7	9.92	14.9	9.64	14.5
V_n/Ω_v	$\phi_v V_n$, kips	199	298	177	265	155	232	183	274	165	248	151	227
Z_x, in.^{3}		211		186		163		146		133		123	
L_p, ft		9.36		9.29		9.22		6.00		5.97		5.93	
L_r, ft		30.3		28.5		27.1		19.6		18.8		18.2	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W18–W16

Shape		W18×										W16×	
		55		50		46		40		35		100	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	6					260	390	226	338	212	319		
	7	283	424	255	383	259	389	224	336	190	285		
	8	279	420	252	379	226	340	196	294	166	249		
	9	248	373	224	337	201	302	174	261	147	222	397	596
	10	224	336	202	303	181	272	156	235	133	200	395	594
	11	203	305	183	275	165	247	142	214	121	181	359	540
	12	186	280	168	253	151	227	130	196	111	166	329	495
	13	172	258	155	233	139	209	120	181	102	153	304	457
	14	160	240	144	216	129	194	112	168	94.8	143	282	424
	15	149	224	134	202	121	181	104	157	88.5	133	263	396
	16	140	210	126	189	113	170	97.8	147	83.0	125	247	371
	17	132	198	119	178	106	160	92.1	138	78.1	117	232	349
	18	124	187	112	168	101	151	86.9	131	73.7	111	220	330
	19	118	177	106	159	95.3	143	82.4	124	69.9	105	208	313
	20	112	168	101	152	90.5	136	78.2	118	66.4	99.8	198	297
	21	106	160	96.0	144	86.2	130	74.5	112	63.2	95.0	188	283
	22	102	153	91.6	138	82.3	124	71.1	107	60.3	90.7	180	270
	23	97.2	146	87.7	132	78.7	118	68.0	102	57.7	86.7	172	258
	24	93.1	140	84.0	126	75.4	113	65.2	98.0	55.3	83.1	165	248
	25	89.4	134	80.6	121	72.4	109	62.6	94.1	53.1	79.8	158	238
	26	86.0	129	77.5	117	69.6	105	60.2	90.5	51.1	76.7	152	228
	27	82.8	124	74.7	112	67.1	101	58.0	87.1	49.2	73.9	146	220
	28	79.8	120	72.0	108	64.7	97.2	55.9	84.0	47.4	71.3	141	212
	29	77.1	116	69.5	104	62.4	93.8	54.0	81.1	45.8	68.8	136	205
	30	74.5	112	67.2	101	60.3	90.7	52.2	78.4	44.2	66.5	132	198
	31	72.1	108	65.0	97.7	58.4	87.8	50.5	75.9	42.8	64.4	127	192
	32	69.9	105	63.0	94.7	56.6	85.0	48.9	73.5	41.5	62.3	124	186
	33	67.7	102	61.1	91.8	54.9	82.5	47.4	71.3	40.2	60.5	120	180
	34	65.8	98.8	59.3	89.1	53.2	80.0	46.0	69.2	39.0	58.7	116	175
	35	63.9	96.0	57.6	86.6	51.7	77.7	44.7	67.2	37.9	57.0	113	170
	36	62.1	93.3	56.0	84.2	50.3	75.6	43.5	65.3	36.9	55.4	110	165
	37	60.4	90.8	54.5	81.9	48.9	73.5	42.3	63.6	35.9	53.9	107	161
	38	58.8	88.4	53.1	79.7	47.6	71.6	41.2	61.9	34.9	52.5	104	156
	39	57.3	86.2	51.7	77.7	46.4	69.8	40.1	60.3	34.0	51.2	101	152
	40	55.9	84.0	50.4	75.8	45.3	68.0	39.1	58.8	33.2	49.9	98.8	149
	42	53.2	80.0	48.0	72.1	43.1	64.8	37.3	56.0	31.6	47.5	94.1	141
	44	50.8	76.4	45.8	68.9	41.1	61.8	35.6	53.5	30.2	45.3		

Beam Properties

		W18×55		W18×50		W18×46		W18×40		W18×35		W16×100	
W_c/Ω_b	$\phi_b W_c$, kip-ft	2240	3360	2020	3030	1810	2720	1560	2350	1330	2000	3950	5940
M_p/Ω_b	$\phi_b M_p$, kip-ft	279	420	252	379	226	340	196	294	166	249	494	743
M_r/Ω_b	$\phi_b M_r$, kip-ft	172	258	155	233	138	207	119	180	101	151	306	459
BF	BF, kips	9.26	13.9	8.69	13.1	9.71	14.6	8.86	13.3	8.07	12.1	7.90	11.9
V_n/Ω_v	$\phi_v V_n$, kips	141	212	128	192	130	195	113	169	106	159	199	298

	W18×55	W18×50	W18×46	W18×40	W18×35	W16×100
Z_x, in.^{3}	112	101	90.7	78.4	66.5	198
L_p, ft	5.90	5.83	4.56	4.49	4.31	8.87
L_r, ft	17.5	17.0	13.7	13.1	12.4	32.7

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W16

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W16×									
		89		77		67		57		50	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	7							283	424	247	371
	8							262	394	230	345
	9	352	528	301	451			233	350	204	307
	10	349	525	299	450	258	387	210	315	184	276
	11	318	477	272	409	236	355	191	286	167	251
	12	291	438	250	375	216	325	175	263	153	230
	13	269	404	230	346	200	300	161	242	141	212
	14	250	375	214	321	185	279	150	225	131	197
	15	233	350	200	300	173	260	140	210	122	184
	16	218	328	187	281	162	244	131	197	115	173
	17	205	309	176	265	153	229	123	185	108	162
	18	194	292	166	250	144	217	116	175	102	153
	19	184	276	158	237	137	205	110	166	96.6	145
	20	175	263	150	225	130	195	105	158	91.8	138
	21	166	250	143	214	124	186	99.8	150	87.4	131
	22	159	239	136	205	118	177	95.3	143	83.5	125
	23	152	228	130	196	113	170	91.1	137	79.8	120
	24	146	219	125	188	108	163	87.3	131	76.5	115
	25	140	210	120	180	104	156	83.8	126	73.5	110
	26	134	202	115	173	99.8	150	80.6	121	70.6	106
	27	129	194	111	167	96.1	144	77.6	117	68.0	102
	28	125	188	107	161	92.7	139	74.9	113	65.6	98.6
	29	120	181	103	155	89.5	134	72.3	109	63.3	95.2
	30	116	175	99.8	150	86.5	130	69.9	105	61.2	92.0
	31	113	169	96.6	145	83.7	126	67.6	102	59.2	89.0
	32	109	164	93.6	141	81.1	122	65.5	98.4	57.4	86.3
	33	106	159	90.7	136	78.6	118	63.5	95.5	55.6	83.6
	34	103	154	88.1	132	76.3	115	61.6	92.6	54.0	81.2
	35	99.8	150	85.5	129	74.1	111	59.9	90.0	52.5	78.9
	36	97.0	146	83.2	125	72.1	108	58.2	87.5	51.0	76.7
	37	94.4	142	80.9	122	70.1	105	56.6	85.1	49.6	74.6
	38	91.9	138	78.8	118	68.3	103	55.2	82.9	48.3	72.6
	39	89.6	135	76.8	115	66.5	100	53.7	80.8	47.1	70.8
	40	87.3	131	74.9	113	64.9	97.5	52.4	78.8	45.9	69.0

Beam Properties

		89 ASD	89 LRFD	77 ASD	77 LRFD	67 ASD	67 LRFD	57 ASD	57 LRFD	50 ASD	50 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	3490	5250	2990	4500	2590	3900	2100	3150	1840	2760
M_p/Ω_b	$\phi_b M_p$, kip-ft	437	656	374	563	324	488	262	394	230	345
M_r/Ω_b	$\phi_b M_r$, kip-ft	271	407	234	352	204	307	161	242	141	213
BF	BF, kips	7.74	11.6	7.34	11.0	6.91	10.4	7.98	12.0	7.59	11.4
V_n/Ω_v	$\phi_v V_n$, kips	176	264	150	225	129	194	141	212	124	185

	89	77	67	57	50
Z_x, in.^{3}	175	150	130	105	92.0
L_p, ft	8.80	8.72	8.69	5.65	5.62
L_r, ft	30.2	27.8	26.1	18.3	17.2

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[v] Shape does not meet the h/t_w limit for shear in Specification Section G2.1a with $F_y = 50$ ksi, $\Omega_v = 1.67$, $\phi_v = 0.90$.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W16

Shape		W16×									
		45		40		36		31		26^v	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	6					187	281	175	262	141	212
	7	223	334	195	293	182	274	154	231	126	189
	8	205	309	182	274	160	240	135	203	110	166
	9	183	274	162	243	142	213	120	180	98.0	147
	10	164	247	146	219	128	192	108	162	88.2	133
	11	149	224	132	199	116	175	98.0	147	80.2	121
	12	137	206	121	183	106	160	89.8	135	73.5	111
	13	126	190	112	168	98.3	148	82.9	125	67.9	102
	14	117	176	104	156	91.2	137	77.0	116	63.0	94.7
	15	110	165	97.1	146	85.2	128	71.9	108	58.8	88.4
	16	103	154	91.1	137	79.8	120	67.4	101	55.1	82.9
	17	96.6	145	85.7	129	75.1	113	63.4	95.3	51.9	78.0
	18	91.3	137	80.9	122	71.0	107	59.9	90.0	49.0	73.7
	19	86.5	130	76.7	115	67.2	101	56.7	85.3	46.4	69.8
	20	82.1	123	72.9	110	63.9	96.0	53.9	81.0	44.1	66.3
	21	78.2	118	69.4	104	60.8	91.4	51.3	77.1	42.0	63.1
	22	74.7	112	66.2	99.5	58.1	87.3	49.0	73.6	40.1	60.3
	23	71.4	107	63.4	95.2	55.5	83.5	46.9	70.4	38.4	57.7
	24	68.4	103	60.7	91.3	53.2	80.0	44.9	67.5	36.8	55.3
	25	65.7	98.8	58.3	87.6	51.1	76.8	43.1	64.8	35.3	53.0
	26	63.2	95.0	56.0	84.2	49.1	73.8	41.5	62.3	33.9	51.0
	27	60.8	91.4	54.0	81.1	47.3	71.1	39.9	60.0	32.7	49.1
	28	58.7	88.2	52.0	78.2	45.6	68.6	38.5	57.9	31.5	47.4
	29	56.6	85.1	50.2	75.5	44.0	66.2	37.2	55.9	30.4	45.7
	30	54.8	82.3	48.6	73.0	42.6	64.0	35.9	54.0	29.4	44.2
	31	53.0	79.6	47.0	70.6	41.2	61.9	34.8	52.3	28.5	42.8
	32	51.3	77.2	45.5	68.4	39.9	60.0	33.7	50.6	27.6	41.4
	33	49.8	74.8	44.2	66.4	38.7	58.2	32.7	49.1	26.7	40.2
	34	48.3	72.6	42.9	64.4	37.6	56.5	31.7	47.6	25.9	39.0
	35	46.9	70.5	41.6	62.6	36.5	54.9	30.8	46.3	25.2	37.9
	36	45.6	68.6	40.5	60.8	35.5	53.3	29.9	45.0	24.5	36.8
	37	44.4	66.7	39.4	59.2	34.5	51.9	29.1	43.8	23.8	35.8
	38	43.2	65.0	38.3	57.6	33.6	50.5	28.4	42.6	23.2	34.9
	39	42.1	63.3	37.4	56.2	32.8	49.2	27.6	41.5	22.6	34.0
	40	41.1	61.7	36.4	54.8						
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	1640	2470	1460	2190	1280	1920	1080	1620	882	1330
M_p/Ω_b	$\phi_b M_p$, kip-ft	205	309	182	274	160	240	135	203	110	166
M_r/Ω_b	$\phi_b M_r$, kip-ft	127	191	113	170	98.7	148	82.4	124	67.1	101
BF	BF, kips	7.16	10.8	6.69	10.1	6.19	9.31	6.76	10.2	5.96	8.96
V_n/Ω_v	$\phi_v V_n$, kips	111	167	97.7	146	93.6	140	87.3	131	70.5	106
Z_x, in.^{3}		82.3		73.0		64.0		54.0		44.2	
L_p, ft		5.55		5.55		5.37		4.13		3.96	
L_r, ft		16.5		15.9		15.2		11.9		11.2	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W14

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W14×											
		730^h		665^h		605^h		550^h		500^h		455^h	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	12	2750	4130	2450	3670	2170	3260	1930	2890	1720	2580	1530	2300
	13	2550	3830	2270	3420	2030	3050	1810	2720	1610	2420	1440	2160
	14	2370	3560	2110	3170	1880	2830	1680	2530	1500	2250	1330	2010
	15	2210	3320	1970	2960	1760	2640	1570	2360	1400	2100	1250	1870
	16	2070	3110	1850	2780	1650	2480	1470	2210	1310	1970	1170	1760
	17	1950	2930	1740	2610	1550	2330	1390	2080	1230	1850	1100	1650
	18	1840	2770	1640	2470	1460	2200	1310	1970	1160	1750	1040	1560
	19	1740	2620	1550	2340	1390	2080	1240	1860	1100	1660	983	1480
	20	1660	2490	1480	2220	1320	1980	1180	1770	1050	1580	934	1400
	21	1580	2370	1410	2110	1250	1890	1120	1690	998	1500	890	1340
	22	1510	2260	1340	2020	1200	1800	1070	1610	953	1430	849	1280
	23	1440	2170	1280	1930	1150	1720	1020	1540	911	1370	812	1220
	24	1380	2080	1230	1850	1100	1650	981	1480	873	1310	778	1170
	25	1330	1990	1180	1780	1050	1580	942	1420	838	1260	747	1120
	26	1270	1920	1140	1710	1010	1520	906	1360	806	1210	719	1080
	27	1230	1840	1090	1640	976	1470	872	1310	776	1170	692	1040
	28	1180	1780	1060	1590	941	1410	841	1260	749	1130	667	1000
	29	1140	1720	1020	1530	909	1370	812	1220	723	1090	644	968
	30	1100	1660	985	1480	878	1320	785	1180	699	1050	623	936
	31	1070	1610	953	1430	850	1280	760	1140	676	1020	603	906
	32	1040	1560	923	1390	823	1240	736	1110	655	984	584	878
	33	1000	1510	895	1350	798	1200	714	1070	635	955	566	851
	34	975	1460	869	1310	775	1160	693	1040	616	926	549	826
	35	947	1420	844	1270	753	1130	673	1010	599	900	534	802
	36	920	1380	821	1230	732	1100	654	983	582	875	519	780
	37	896	1350	798	1200	712	1070	637	957	566	851	505	759
	38	872	1310	777	1170	693	1040	620	932	552	829	492	739
	39	850	1280	757	1140	676	1020	604	908	537	808	479	720
	40	828	1250	739	1110	659	990	589	885	524	788	467	702
	42	789	1190	703	1060	627	943	561	843	499	750	445	669
	44	753	1130	671	1010	599	900	535	805	476	716	425	638
	46	720	1080	642	965	573	861	512	770	456	685	406	610
	48	690	1040	615	925	549	825	491	738	437	656		
	50	663	996	591	888	527	792	471	708				
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	33100	49800	29500	44400	26300	39600	23600	35400	21000	31500	18700	28100
M_p/Ω_b	$\phi_b M_p$, kip-ft	4140	6230	3690	5550	3290	4950	2940	4430	2620	3940	2340	3510
M_r/Ω_b	$\phi_b M_r$, kip-ft	2240	3360	2010	3020	1820	2730	1630	2440	1460	2200	1320	1980
BF	BF, kips	7.37	11.1	7.12	10.7	6.83	10.3	6.67	10.0	6.42	9.65	6.20	9.31
V_n/Ω_v	$\phi_v V_n$, kips	1380	2060	1220	1840	1090	1630	963	1450	858	1290	767	1150
Z_x, in.^{3}		1660		1480		1320		1180		1050		936	
L_p, ft		16.6		16.3		16.1		15.9		15.6		15.5	
L_r, ft		275		253		232		213		196		179	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W14

Shape		W14×											
		426^h		398^h		370^h		342^h		311^h		283^h	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	12	1400	2100	1290	1940	1190	1780	1080	1620	966	1450	864	1300
	13	1330	2010	1230	1850	1130	1700	1030	1550	926	1390	832	1250
	14	1240	1860	1140	1720	1050	1580	958	1440	860	1290	773	1160
	15	1160	1740	1070	1600	979	1470	894	1340	802	1210	721	1080
	16	1080	1630	999	1500	918	1380	838	1260	752	1130	676	1020
	17	1020	1530	940	1410	864	1300	789	1190	708	1060	636	956
	18	964	1450	888	1340	816	1230	745	1120	669	1000	601	903
	19	913	1370	841	1260	773	1160	706	1060	633	952	569	856
	20	867	1300	799	1200	735	1100	671	1010	602	905	541	813
	21	826	1240	761	1140	700	1050	639	960	573	861	515	774
	22	788	1190	727	1090	668	1000	610	916	547	822	492	739
	23	754	1130	695	1040	639	960	583	877	523	787	470	707
	24	723	1090	666	1000	612	920	559	840	501	754	451	678
	25	694	1040	640	961	588	883	537	806	481	724	433	650
	26	667	1000	615	924	565	849	516	775	463	696	416	625
	27	642	966	592	890	544	818	497	747	446	670	401	602
	28	619	931	571	858	525	789	479	720	430	646	386	581
	29	598	899	551	829	507	761	463	695	415	624	373	561
	30	578	869	533	801	490	736	447	672	401	603	361	542
	31	560	841	516	775	474	712	433	650	388	584	349	525
	32	542	815	500	751	459	690	419	630	376	565	338	508
	33	526	790	484	728	445	669	406	611	365	548	328	493
	34	510	767	470	707	432	649	395	593	354	532	318	478
	35	496	745	457	687	420	631	383	576	344	517	309	465
	36	482	724	444	668	408	613	373	560	334	502	301	452
	37	469	705	432	649	397	597	363	545	325	489	292	439
	38	456	686	421	632	387	581	353	531	317	476	285	428
	39	445	668	410	616	377	566	344	517	309	464	277	417
	40	434	652	400	601	367	552	335	504	301	452	270	407
	42	413	621	381	572	350	526	319	480	287	431		
	44	394	593	363	546	334	502						
	46	377	567										
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	17300	26100	16000	24000	14700	22100	13400	20200	12000	18100	10800	16300
M_p/Ω_b	$\phi_b M_p$, kip-ft	2170	3260	2000	3000	1840	2760	1680	2520	1500	2260	1350	2030
M_r/Ω_b	$\phi_b M_r$, kip-ft	1230	1850	1150	1720	1060	1590	975	1460	884	1330	802	1200
BF	BF, kips	6.09	9.16	5.96	8.96	5.86	8.80	5.75	8.64	5.63	8.46	5.53	8.31
V_n/Ω_v	$\phi_v V_n$, kips	700	1050	647	971	593	890	540	810	483	724	432	648
Z_x, in.^{3}		869		801		736		672		603		542	
L_p, ft		15.3		15.2		15.1		15.0		14.8		14.7	
L_r, ft		169		158		148		137		125		114	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W14

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W14×											
		257		233		211		193		176		159	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	12	770	1150	687	1030	616	924	551	827	505	758	446	670
	13	748	1120	669	1010	599	900	545	819	491	738	441	662
	14	694	1040	622	934	556	836	506	761	456	686	409	615
	15	648	974	580	872	519	780	472	710	426	640	382	574
	16	608	913	544	818	487	731	443	666	399	600	358	538
	17	572	859	512	769	458	688	417	626	376	565	337	506
	18	540	812	483	727	432	650	394	592	355	533	318	478
	19	512	769	458	688	410	616	373	561	336	505	302	453
	20	486	731	435	654	389	585	354	533	319	480	286	431
	21	463	696	414	623	371	557	337	507	304	457	273	410
	22	442	664	396	595	354	532	322	484	290	436	260	391
	23	423	635	378	569	338	509	308	463	278	417	249	374
	24	405	609	363	545	324	488	295	444	266	400	239	359
	25	389	584	348	523	311	468	283	426	255	384	229	344
	26	374	562	335	503	299	450	273	410	246	369	220	331
	27	360	541	322	484	288	433	262	394	237	356	212	319
	28	347	522	311	467	278	418	253	380	228	343	205	308
	29	335	504	300	451	268	403	244	367	220	331	198	297
	30	324	487	290	436	259	390	236	355	213	320	191	287
	31	314	471	281	422	251	377	229	344	206	310	185	278
	32	304	457	272	409	243	366	221	333	200	300	179	269
	33	295	443	264	396	236	355	215	323	194	291	174	261
	34	286	430	256	385	229	344	208	313	188	282	168	253
	35	278	417	249	374	222	334	202	304	182	274	164	246
	36	270	406	242	363	216	325	197	296	177	267	159	239
	37	263	395	235	354	210	316	192	288	173	259	155	233
	38	256	384	229	344	205	308	186	280	168	253		
	39	249	375	223	335	200	300						
	40	243	365	218	327								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	9720	14600	8700	13100	7780	11700	7090	10700	6390	9600	5730	8610
M_p/Ω_b	$\phi_b M_p$, kip-ft	1220	1830	1090	1640	973	1460	886	1330	798	1200	716	1080
M_r/Ω_b	$\phi_b M_r$, kip-ft	725	1090	655	984	590	887	541	814	491	738	444	667
BF	BF, kips	5.46	8.21	5.38	8.09	5.31	7.99	5.27	7.92	5.22	7.84	5.18	7.79
V_n/Ω_v	$\phi_v V_n$, kips	385	577	343	515	308	462	276	413	253	379	223	335
Z_x, in.^{3}		487		436		390		355		320		287	
L_p, ft		14.6		14.5		14.4		14.3		14.2		14.1	
L_r, ft		104		94.9		86.4		79.7		73.2		66.7	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W14

Shape		W14×											
		145		132		120		109		99^f		90^f	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	12	402	603	378	567	342	513	301	451	275	412	247	370
	13	399	600	359	540	326	489	295	443	264	397	235	353
	14	371	557	334	501	302	454	274	411	246	369	218	328
	15	346	520	311	468	282	424	255	384	229	344	203	306
	16	324	488	292	439	264	398	240	360	215	323	191	287
	17	305	459	275	413	249	374	225	339	202	304	180	270
	18	288	433	259	390	235	353	213	320	191	287	170	255
	19	273	411	246	369	223	335	202	303	181	272	161	241
	20	259	390	234	351	212	318	192	288	172	258	153	229
	21	247	371	222	334	202	303	182	274	164	246	145	218
	22	236	355	212	319	192	289	174	262	156	235	139	209
	23	226	339	203	305	184	277	167	250	149	225	133	199
	24	216	325	195	293	176	265	160	240	143	215	127	191
	25	208	312	187	281	169	254	153	230	137	207	122	183
	26	200	300	180	270	163	245	147	222	132	199	117	176
	27	192	289	173	260	157	236	142	213	127	191	113	170
	28	185	279	167	251	151	227	137	206	123	185	109	164
	29	179	269	161	242	146	219	132	199	119	178	105	158
	30	173	260	156	234	141	212	128	192	115	172	102	153
	31	167	252	151	226	137	205	124	186	111	167	98.5	148
	32	162	244	146	219	132	199	120	180	107	161	95.4	143
	33	157	236	142	213	128	193	116	175	104	157	92.5	139
	34	153	229	137	206	124	187	113	169	101	152	89.8	135
	35	148	223	133	201	121	182	109	165	98.2	148	87.2	131
	36	144	217	130	195	118	177						
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	5190	7800	4670	7020	4230	6360	3830	5760	3440	5170	3050	4590
M_p/Ω_b	$\phi_b M_p$, kip-ft	649	975	584	878	529	795	479	720	430	646	382	573
M_r/Ω_b	$\phi_b M_r$, kip-ft	405	609	365	549	332	499	302	454	274	412	250	375
BF	BF, kips	5.11	7.68	5.13	7.70	5.09	7.64	5.02	7.54	4.89	7.35	4.80	7.22
V_n/Ω_v	$\phi_v V_n$, kips	201	302	189	284	171	256	150	226	137	206	123	185
Z_x, in.^{3}		260		234		212		192		173		157	
L_p, ft		14.1		13.3		13.2		13.2		13.5		15.2	
L_r, ft		61.7		56.0		52.0		48.4		45.3		42.6	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W14

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W14×											
		82		74		68		61		53		48	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	8									206	309	188	281
	9	292	438	255	383	233	350	208	313	193	290	174	261
	10	277	417	251	378	230	345	204	306	174	261	156	235
	11	252	379	229	344	209	314	185	278	158	238	142	214
	12	231	348	210	315	191	288	170	255	145	218	130	196
	13	213	321	193	291	177	265	157	235	134	201	120	181
	14	198	298	180	270	164	246	145	219	124	187	112	168
	15	185	278	168	252	153	230	136	204	116	174	104	157
	16	173	261	157	236	143	216	127	191	109	163	97.8	147
	17	163	245	148	222	135	203	120	180	102	154	92.1	138
	18	154	232	140	210	128	192	113	170	96.6	145	86.9	131
	19	146	219	132	199	121	182	107	161	91.5	138	82.4	124
	20	139	209	126	189	115	173	102	153	86.9	131	78.2	118
	21	132	199	120	180	109	164	96.9	146	82.8	124	74.5	112
	22	126	190	114	172	104	157	92.5	139	79.0	119	71.1	107
	23	121	181	109	164	99.8	150	88.5	133	75.6	114	68.0	102
	24	116	174	105	158	95.6	144	84.8	128	72.4	109	65.2	98.0
	25	111	167	101	151	91.8	138	81.4	122	69.5	105	62.6	94.1
	26	107	160	96.7	145	88.3	133	78.3	118	66.9	101	60.2	90.5
	27	103	154	93.1	140	85.0	128	75.4	113	64.4	96.8	58.0	87.1
	28	99.1	149	89.8	135	82.0	123	72.7	109	62.1	93.3	55.9	84.0
	29	95.7	144	86.7	130	79.2	119	70.2	106	59.9	90.1	54.0	81.1
	30	92.5	139	83.8	126	76.5	115	67.9	102	58.0	87.1	52.2	78.4
	31	89.5	135	81.1	122	74.0	111	65.7	98.7	56.1	84.3	50.5	75.9
	32	86.7	130	78.6	118	71.7	108	63.6	95.6	54.3	81.7	48.9	73.5
	33	84.1	126	76.2	115	69.6	105	61.7	92.7	52.7	79.2	47.4	71.3
	34	81.6	123	74.0	111	67.5	101	59.9	90.0	51.1	76.9	46.0	69.2
	35	79.3	119	71.9	108	65.6	98.6						
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	2770	4170	2510	3780	2300	3450	2040	3060	1740	2610	1560	2350
M_p/Ω_b	$\phi_b M_p$, kip-ft	347	521	314	473	287	431	254	383	217	327	196	294
M_r/Ω_b	$\phi_b M_r$, kip-ft	215	323	196	294	180	270	161	242	136	204	123	184
BF	BF, kips	5.43	8.16	5.34	8.03	5.20	7.81	4.96	7.46	5.27	7.93	5.10	7.66
V_n/Ω_v	$\phi_v V_n$, kips	146	219	128	191	117	175	104	156	103	155	93.8	141
Z_x, in.^{3}		139		126		115		102		87.1		78.4	
L_p, ft		8.76		8.76		8.69		8.65		6.78		6.75	
L_r, ft		33.1		31.0		29.3		27.5		22.2		21.1	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

F_y = 50 ksi

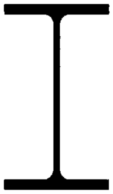
Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W14

Shape		**W14×**											
		43		**38**		**34**		**30**		**26**		**22**	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Span, ft	**5**									142	213	126	190
	6					159	239	149	224	134	201	110	166
	7			175	262	156	234	135	203	115	172	94.7	142
	8	167	250	153	231	136	205	118	177	100	151	82.8	125
	9	154	232	136	205	121	182	105	158	89.2	134	73.6	111
	10	139	209	123	185	109	164	94.4	142	80.2	121	66.3	99.6
	11	126	190	112	168	99.1	149	85.8	129	72.9	110	60.2	90.5
	12	116	174	102	154	90.8	137	78.7	118	66.9	101	55.2	83.0
	13	107	161	94.4	142	83.8	126	72.6	109	61.7	92.8	51.0	76.6
	14	99.2	149	87.7	132	77.8	117	67.4	101	57.3	86.1	47.3	71.1
	15	92.6	139	81.8	123	72.7	109	62.9	94.6	53.5	80.4	44.2	66.4
	16	86.8	130	76.7	115	68.1	102	59.0	88.7	50.1	75.4	41.4	62.3
	17	81.7	123	72.2	109	64.1	96.4	55.5	83.5	47.2	70.9	39.0	58.6
	18	77.2	116	68.2	103	60.5	91.0	52.5	78.8	44.6	67.0	36.8	55.3
	19	73.1	110	64.6	97.1	57.4	86.2	49.7	74.7	42.2	63.5	34.9	52.4
	20	69.5	104	61.4	92.3	54.5	81.9	47.2	71.0	40.1	60.3	33.1	49.8
	21	66.2	99.4	58.5	87.9	51.9	78.0	45.0	67.6	38.2	57.4	31.6	47.4
	22	63.1	94.9	55.8	83.9	49.5	74.5	42.9	64.5	36.5	54.8	30.1	45.3
	23	60.4	90.8	53.4	80.2	47.4	71.2	41.0	61.7	34.9	52.4	28.8	43.3
	24	57.9	87.0	51.1	76.9	45.4	68.3	39.3	59.1	33.4	50.3	27.6	41.5
	25	55.6	83.5	49.1	73.8	43.6	65.5	37.8	56.8	32.1	48.2	26.5	39.8
	26	53.4	80.3	47.2	71.0	41.9	63.0	36.3	54.6	30.9	46.4	25.5	38.3
	27	51.5	77.3	45.5	68.3	40.4	60.7	35.0	52.6	29.7	44.7	24.5	36.9
	28	49.6	74.6	43.8	65.9	38.9	58.5	33.7	50.7	28.7	43.1	23.7	35.6
	29	47.9	72.0	42.3	63.6	37.6	56.5	32.6	48.9	27.7	41.6	22.9	34.3
	30	46.3	69.6	40.9	61.5	36.3	54.6	31.5	47.3	26.7	40.2	22.1	33.2
	31	44.8	67.4	39.6	59.5	35.2	52.8	30.5	45.8	25.9	38.9	21.4	32.1
	32	43.4	65.2	38.4	57.7	34.1	51.2	29.5	44.3	25.1	37.7	20.7	31.1
	33	42.1	63.3	37.2	55.9	33.0	49.6	28.6	43.0	24.3	36.5	20.1	30.2
	34	40.9	61.4	36.1	54.3	32.1	48.2	27.8	41.7	23.6	35.5	19.5	29.3
	35			35.1	52.7								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	1390	2090	1230	1850	1090	1640	944	1420	802	1210	663	996
M_p/Ω_b	$\phi_b M_p$, kip-ft	174	261	153	231	136	205	118	177	100	151	82.8	125
M_r/Ω_b	$\phi_b M_r$, kip-ft	109	164	95.4	143	84.9	128	73.4	110	61.7	92.7	50.6	76.1
BF	BF, kips	4.82	7.24	5.39	8.10	5.05	7.59	4.65	6.99	5.32	7.99	4.75	7.14
V_n/Ω_v	$\phi_v V_n$, kips	83.3	125	87.4	131	79.7	120	74.7	112	70.9	106	63.2	94.8
Z_x, in.^{3}		69.6		61.5		54.6		47.3		40.2		33.2	
L_p, ft		6.68		5.47		5.40		5.26		3.81		3.67	
L_r, ft		20.0		16.2		15.6		14.9		11.1		10.4	

ASD	**LRFD**
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.



W12

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

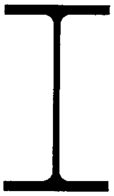
Shape		W12×											
		336[h]		305[h]		279[h]		252[h]		230[h]		210	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	9					970	1460	860	1290	774			
	10	1190	1790	1060	1590	960	1440	854	1280	770	1160	694	1040
	11	1090	1640	974	1460	873	1310	777	1170	700	1050	631	949
	12	1000	1510	893	1340	800	1200	712	1070	642	965	579	870
	13	926	1390	825	1240	739	1110	657	988	593	891	534	803
	14	860	1290	766	1150	686	1030	610	917	550	827	496	746
	15	802	1210	715	1070	640	962	570	856	514	772	463	696
	16	752	1130	670	1010	600	902	534	803	482	724	434	653
	17	708	1060	631	948	565	849	503	755	453	681	409	614
	18	669	1000	595	895	533	802	475	713	428	643	386	580
	19	633	952	564	848	505	759	450	676	406	609	366	549
	20	602	905	536	806	480	722	427	642	385	579	347	522
	21	573	861	510	767	457	687	407	611	367	551	331	497
	22	547	822	487	732	436	656	388	584	350	526	316	475
	23	523	787	466	700	417	627	371	558	335	503	302	454
	24	501	754	447	671	400	601	356	535	321	483	289	435
	25	481	724	429	644	384	577	342	514	308	463	278	418
	26	463	696	412	620	369	555	329	494	296	445	267	402
	27	446	670	397	597	356	534	316	476	285	429	257	387
	28	430	646	383	575	343	515	305	459	275	414	248	373
	29	415	624	370	556	331	498	295	443	266	399	240	360
	30	401	603	357	537	320	481	285	428	257	386	232	348
	31	388	584	346	520	310	465	276	414	249	374	224	337
	32	376	565	335	503	300	451	267	401	241	362	217	326
	33	365	548	325	488	291	437	259	389	233	351	210	316
	34	354	532	315	474	282	424	251	378	227	341	204	307
	35	344	517	306	460	274	412	244	367	220	331	198	298
	36	334	502	298	447	267	401	237	357	214	322	193	290
	37	325	489	290	435	259	390	231	347	208	313		
	38	317	476	282	424	253	380	225	338				
	39	309	464	275	413	246	370						
	40	301	452	268	403								
	41	294	441										
	42	287	431										
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	12000	18100	10700	16100	9600	14400	8540	12800	7700	11600	6950	10400
M_p/Ω_b	$\phi_b M_p$, kip-ft	1500	2260	1340	2010	1200	1800	1070	1610	963	1450	868	1310
M_r/Ω_b	$\phi_b M_r$, kip-ft	844	1270	760	1140	686	1030	617	927	561	843	510	767
BF	BF, kips	4.80	7.22	4.66	7.00	4.52	6.79	4.40	6.62	4.32	6.49	4.24	6.38
V_n/Ω_v	$\phi_v V_n$, kips	597	896	530	796	485	728	430	645	387	580	347	521
Z_x, in.^{3}		603		537		481		428		386		348	
L_p, ft		12.3		12.1		11.9		11.8		11.7		11.6	
L_r, ft		150		137		126		114		105		96.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes


W12

Shape		W12×											
		190		170		152		136		120		106	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	9									**373**	559		
	10	**610**	915	**539**	808	**477**	716	**424**	636	**371**	558	**315**	472
	11	**564**	848	**499**	750	**441**	663	**388**	584	**338**	507	**298**	447
	12	**517**	777	**457**	687	**404**	608	**356**	535	**309**	465	**273**	410
	13	**478**	718	**422**	635	**373**	561	**329**	494	**286**	429	**252**	378
	14	**443**	666	**392**	589	**346**	521	**305**	459	**265**	399	**234**	351
	15	**414**	622	**366**	550	**323**	486	**285**	428	**248**	372	**218**	328
	16	**388**	583	**343**	516	**303**	456	**267**	401	**232**	349	**205**	308
	17	**365**	549	**323**	485	**285**	429	**251**	378	**218**	328	**193**	289
	18	**345**	518	**305**	458	**269**	405	**237**	357	**206**	310	**182**	273
	19	**327**	491	**289**	434	**255**	384	**225**	338	**195**	294	**172**	259
	20	**310**	467	**274**	413	**243**	365	**214**	321	**186**	279	**164**	246
	21	**296**	444	**261**	393	**231**	347	**203**	306	**177**	266	**156**	234
	22	**282**	424	**250**	375	**220**	331	**194**	292	**169**	254	**149**	224
	23	**270**	406	**239**	359	**211**	317	**186**	279	**161**	243	**142**	214
	24	**259**	389	**229**	344	**202**	304	**178**	267	**155**	233	**136**	205
	25	**248**	373	**220**	330	**194**	292	**171**	257	**149**	223	**131**	197
	26	**239**	359	**211**	317	**187**	280	**164**	247	**143**	215	**126**	189
	27	**230**	346	**203**	306	**180**	270	**158**	238	**138**	207	**121**	182
	28	**222**	333	**196**	295	**173**	260	**153**	229	**133**	199	**117**	176
	29	**214**	322	**189**	284	**167**	251	**147**	221	**128**	192	**113**	170
	30	**207**	311	**183**	275	**162**	243	**142**	214	**124**	186	**109**	164
	31	**200**	301	**177**	266	**156**	235	**138**	207	**120**	180	**106**	159
	32	**194**	292	**172**	258	**152**	228	**133**	201	**116**	174	**102**	154
	33	**188**	283	**166**	250	**147**	221	**129**	195				
	34	**183**	274	**161**	243	**143**	214						
	35	**177**	267	**157**	236								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	**6210**	9330	**5490**	8250	**4850**	7290	**4270**	6420	**3710**	5580	**3270**	4920
M_p/Ω_b	$\phi_b M_p$, kip-ft	**776**	1170	**686**	1030	**606**	911	**534**	803	**464**	698	**409**	615
M_r/Ω_b	$\phi_b M_r$, kip-ft	**459**	690	**410**	617	**365**	549	**325**	488	**285**	428	**253**	381
BF	*BF*, kips	**4.18**	6.28	**4.11**	6.18	**4.07**	6.11	**4.01**	6.03	**3.95**	5.93	**3.93**	5.90
V_n/Ω_v	$\phi_v V_n$, kips	**305**	457	**269**	404	**239**	358	**212**	318	**186**	279	**157**	236
Z_x, in.^{3}		311		275		243		214		186		164	
L_p, ft		11.5		11.4		11.3		11.2		11.1		11.0	
L_r, ft		87.3		78.5		70.6		63.3		56.5		50.7	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W12

Table 3–6 (continued)

Maximum Total Uniform Load, kips

W Shapes

F_y = 50 ksi

Shape		W12×											
		96		87		79		72		65^f		58	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	9											176	263
	10	280	419	258	387	233	349	211	316	189	284	172	259
	11	267	401	240	360	216	325	196	295	172	259	157	236
	12	245	368	220	330	198	298	180	270	158	237	144	216
	13	226	339	203	305	183	275	166	249	146	219	133	199
	14	210	315	188	283	170	255	154	231	135	204	123	185
	15	196	294	176	264	158	238	144	216	126	190	115	173
	16	183	276	165	248	148	223	135	203	119	178	108	162
	17	173	259	155	233	140	210	127	191	112	168	101	152
	18	163	245	146	220	132	198	120	180	105	158	95.8	144
	19	154	232	139	208	125	188	113	171	99.8	150	90.8	136
	20	147	221	132	198	119	179	108	162	94.8	142	86.2	130
	21	140	210	125	189	113	170	103	154	90.3	136	82.1	123
	22	133	200	120	180	108	162	98.0	147	86.2	130	78.4	118
	23	128	192	115	172	103	155	93.7	141	82.4	124	75.0	113
	24	122	184	110	165	99.0	149	89.8	135	79.0	119	71.9	108
	25	117	176	105	158	95.0	143	86.2	130	75.8	114	69.0	104
	26	113	170	101	152	91.4	137	82.9	125	72.9	110	66.3	99.7
	27	109	163	97.6	147	88.0	132	79.8	120	70.2	106	63.9	96.0
	28	105	158	94.1	141	84.8	128	77.0	116	67.7	102	61.6	92.6
	29	101	152	90.9	137	81.9	123	74.3	112	65.4	98.3	59.5	89.4
	30	97.8	147	87.8	132	79.2	119	71.9	108	63.2	95.0	57.5	86.4
	31	94.6	142	85.0	128								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	2930	4410	2630	3960	2380	3570	2160	3240	1900	2850	1720	2590
M_p/Ω_b	$\phi_b M_p$, kip-ft	367	551	329	495	297	446	269	405	237	356	216	324
M_r/Ω_b	$\phi_b M_r$, kip-ft	229	344	206	310	187	281	170	256	154	231	136	205
BF	BF, kips	3.87	5.81	3.84	5.76	3.77	5.67	3.72	5.59	3.60	5.41	3.76	5.66
V_n/Ω_v	$\phi_v V_n$, kips	140	210	129	194	116	175	105	158	94.5	142	87.8	132
Z_x, in.^{3}		147		132		119		108		96.8		86.4	
L_p, ft		10.9		10.8		10.8		10.7		11.9		8.87	
L_r, ft		46.6		43.0		39.9		37.4		35.1		29.9	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

$F_y = 50$ ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W12

Shape	W12×											
	53		50		45		40		35		30	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft 6									150	225	128	193
7			180	271	162	242			146	219	123	185
8			179	270	160	241	141	211	128	192	108	162
9	166	250	159	240	142	214	126	190	114	171	95.6	144
10	155	234	144	216	128	193	114	171	102	154	86.0	129
11	141	212	130	196	116	175	103	155	92.9	140	78.2	118
12	130	195	120	180	107	161	94.8	143	85.2	128	71.7	108
13	120	180	110	166	98.6	148	87.5	132	78.6	118	66.2	99.5
14	111	167	103	154	91.5	138	81.3	122	73.0	110	61.4	92.4
15	104	156	95.7	144	85.4	128	75.8	114	68.1	102	57.4	86.2
16	97.2	146	89.7	135	80.1	120	71.1	107	63.9	96.0	53.8	80.8
17	91.5	137	84.4	127	75.4	113	66.9	101	60.1	90.4	50.6	76.1
18	86.4	130	79.7	120	71.2	107	63.2	95.0	56.8	85.3	47.8	71.8
19	81.8	123	75.5	114	67.4	101	59.9	90.0	53.8	80.8	45.3	68.1
20	77.7	117	71.8	108	64.1	96.3	56.9	85.5	51.1	76.8	43.0	64.7
21	74.0	111	68.3	103	61.0	91.7	54.2	81.4	48.7	73.1	41.0	61.6
22	70.7	106	65.2	98.0	58.2	87.5	51.7	77.7	46.5	69.8	39.1	58.8
23	67.6	102	62.4	93.8	55.7	83.7	49.5	74.3	44.4	66.8	37.4	56.2
24	64.8	97.4	59.8	89.9	53.4	80.3	47.4	71.3	42.6	64.0	35.8	53.9
25	62.2	93.5	57.4	86.3	51.3	77.0	45.5	68.4	40.9	61.4	34.4	51.7
26	59.8	89.9	55.2	83.0	49.3	74.1	43.8	65.8	39.3	59.1	33.1	49.7
27	57.6	86.6	53.2	79.9	47.5	71.3	42.1	63.3	37.9	56.9	31.9	47.9
28	55.5	83.5	51.3	77.0	45.8	68.8	40.6	61.1	36.5	54.9	30.7	46.2
29	53.6	80.6	49.5	74.4	44.2	66.4	39.2	59.0	35.2	53.0	29.7	44.6
30	51.8	77.9	47.8	71.9	42.7	64.2			34.1	51.2	28.7	43.1
31									33.0	49.5		

Beam Properties

ASD	LRFD	53 ASD	53 LRFD	50 ASD	50 LRFD	45 ASD	45 LRFD	40 ASD	40 LRFD	35 ASD	35 LRFD	30 ASD	30 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	1550	2340	1440	2160	1280	1930	1140	1710	1020	1540	860	1290
M_p/Ω_b	$\phi_b M_p$, kip-ft	194	292	179	270	160	241	142	214	128	192	108	162
M_r/Ω_b	$\phi_b M_r$, kip-ft	123	185	112	169	101	151	89.9	135	79.6	120	67.4	101
BF	*BF*, kips	3.65	5.48	3.97	5.97	3.83	5.75	3.66	5.50	4.28	6.43	3.92	5.89
V_n/Ω_v	$\phi_v V_n$, kips	83.2	125	90.2	135	80.8	121	70.4	106	75.0	113	64.2	96.3

	53	50	45	40	35	30
Z_x, in.^{3}	77.9	71.9	64.2	57.0	51.2	43.1
L_p, ft	8.76	6.92	6.89	6.85	5.44	5.37
L_r, ft	28.2	23.9	22.4	21.1	16.7	15.6

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W12

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W12×										W10×	
		26		22		19		16		14^f		112	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	3							106	158				
	4			128	192	114	171	100	151	85.6	129		
	5			117	176	98.6	148	80.2	121	69.5	104		
	6	112	169	97.5	147	82.2	124	66.9	101	57.9	87.0		
	7	106	159	83.5	126	70.4	106	57.3	86.1	49.6	74.6		
	8	92.8	140	73.1	110	61.6	92.6	50.1	75.4	43.4	65.2	343	515
	9	82.5	124	65.0	97.7	54.8	82.3	44.6	67.0	38.6	58.0	326	490
	10	74.3	112	58.5	87.9	49.3	74.1	40.1	60.3	34.7	52.2	293	441
	11	67.5	101	53.2	79.9	44.8	67.4	36.5	54.8	31.6	47.5	267	401
	12	61.9	93.0	48.7	73.3	41.1	61.8	33.4	50.3	28.9	43.5	245	368
	13	57.1	85.8	45.0	67.6	37.9	57.0	30.9	46.4	26.7	40.2	226	339
	14	53.0	79.7	41.8	62.8	35.2	52.9	28.7	43.1	24.8	37.3	210	315
	15	49.5	74.4	39.0	58.6	32.9	49.4	26.7	40.2	23.2	34.8	196	294
	16	46.4	69.8	36.6	54.9	30.8	46.3	25.1	37.7	21.7	32.6	183	276
	17	43.7	65.6	34.4	51.7	29.0	43.6	23.6	35.5	20.4	30.7	173	259
	18	41.3	62.0	32.5	48.8	27.4	41.2	22.3	33.5	19.3	29.0	163	245
	19	39.1	58.7	30.8	46.3	25.9	39.0	21.1	31.7	18.3	27.5	154	232
	20	37.1	55.8	29.2	44.0	24.7	37.1	20.1	30.2	17.4	26.1	147	221
	21	35.4	53.1	27.8	41.9	23.5	35.3	19.1	28.7	16.5	24.9	140	210
	22	33.8	50.7	26.6	40.0	22.4	33.7	18.2	27.4	15.8	23.7	133	200
	23	32.3	48.5	25.4	38.2	21.4	32.2	17.4	26.2	15.1	22.7	128	192
	24	30.9	46.5	24.4	36.6	20.5	30.9	16.7	25.1	14.5	21.7	122	184
	25	29.7	44.6	23.4	35.2	19.7	29.6	16.0	24.1	13.9	20.9	117	176
	26	28.6	42.9	22.5	33.8	19.0	28.5	15.4	23.2	13.4	20.1	113	170
	27	27.5	41.3	21.7	32.6	18.3	27.4	14.9	22.3	12.9	19.3	109	163
	28	26.5	39.9	20.9	31.4	17.6	26.5	14.3	21.5	12.4	18.6	105	158
	29	25.6	38.5	20.2	30.3	17.0	25.6	13.8	20.8	12.0	18.0		
	30	24.8	37.2	19.5	29.3	16.4	24.7						

Beam Properties

		26 ASD	26 LRFD	22 ASD	22 LRFD	19 ASD	19 LRFD	16 ASD	16 LRFD	14^f ASD	14^f LRFD	112 ASD	112 LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	743	1120	585	879	493	741	401	603	347	522	2930	4410
M_p/Ω_b	$\phi_b M_p$, kip-ft	92.8	140	73.1	110	61.6	92.6	50.1	75.4	43.4	65.2	367	551
M_r/Ω_b	$\phi_b M_r$, kip-ft	58.3	87.7	44.4	66.7	37.2	55.9	29.9	44.9	26.0	39.1	220	331
BF	BF, kips	3.61	5.42	4.65	6.99	4.27	6.43	3.82	5.75	3.42	5.15	2.68	4.02
V_n/Ω_v	$\phi_v V_n$, kips	56.2	84.3	64.0	96.0	57.2	85.7	52.8	79.1	42.8	64.3	172	257

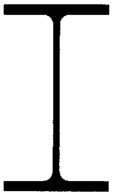
	26	22	19	16	14^f	112
Z_x, in.^{3}	37.2	29.3	24.7	20.1	17.4	147
L_p, ft	5.33	3.00	2.90	2.73	2.66	9.47
L_r, ft	14.9	9.17	8.62	8.03	7.74	64.3

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

^{f} Shape does not meet compact limit for flexure with F_y = 50 ksi.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

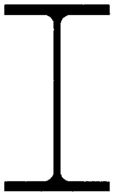


W10

Shape		W10×											
		100		88		77		68		60		54	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	8	**302**	453	**262**	393	**225**	337	**196**	293	**172**	258	**149**	224
	9	**288**	433	**251**	377	**216**	325	**189**	284	**165**	249	**148**	222
	10	**259**	390	**226**	339	**195**	293	**170**	256	**149**	224	**133**	200
	11	**236**	355	**205**	308	**177**	266	**155**	233	**135**	203	**121**	182
	12	**216**	325	**188**	283	**162**	244	**142**	213	**124**	186	**111**	166
	13	**200**	300	**173**	261	**150**	225	**131**	197	**115**	172	**102**	154
	14	**185**	279	**161**	242	**139**	209	**122**	183	**106**	160	**95.0**	143
	15	**173**	260	**150**	226	**130**	195	**114**	171	**99.3**	149	**88.6**	133
	16	**162**	244	**141**	212	**122**	183	**106**	160	**93.1**	140	**83.1**	125
	17	**153**	229	**133**	199	**115**	172	**100**	151	**87.6**	132	**78.2**	118
	18	**144**	217	**125**	188	**108**	163	**94.6**	142	**82.7**	124	**73.9**	111
	19	**137**	205	**119**	178	**103**	154	**89.6**	135	**78.4**	118	**70.0**	105
	20	**130**	195	**113**	170	**97.4**	146	**85.1**	128	**74.5**	112	**66.5**	99.9
	21	**124**	186	**107**	161	**92.8**	139	**81.1**	122	**70.9**	107	**63.3**	95.1
	22	**118**	177	**103**	154	**88.6**	133	**77.4**	116	**67.7**	102	**60.4**	90.8
	23	**113**	170	**98.1**	147	**84.7**	127	**74.0**	111	**64.7**	97.3	**57.8**	86.9
	24	**108**	163	**94.0**	141	**81.2**	122	**70.9**	107	**62.0**	93.2	**55.4**	83.2
	25	**104**	156	**90.2**	136	**77.9**	117	**68.1**	102	**59.6**	89.5	**53.2**	79.9
	26	**99.8**	150	**86.7**	130	**74.9**	113	**65.5**	98.4				
	27	**96.1**	144	**83.5**	126								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	**2590**	3900	**2260**	3390	**1950**	2930	**1700**	2560	**1490**	2240	**1330**	2000
M_p/Ω_b	$\phi_b M_p$, kip-ft	**324**	488	**282**	424	**244**	366	**213**	320	**186**	280	**166**	250
M_r/Ω_b	$\phi_b M_r$, kip-ft	**196**	294	**172**	259	**150**	225	**132**	199	**116**	175	**105**	158
BF	BF, kips	**2.66**	4.01	**2.63**	3.95	**2.59**	3.90	**2.57**	3.86	**2.53**	3.80	**2.49**	3.74
V_n/Ω_v	$\phi_v V_n$, kips	**151**	226	**131**	197	**112**	169	**97.8**	147	**85.8**	129	**74.7**	112
Z_x, in.^{3}		130		113		97.6		85.3		74.6		66.6	
L_p, ft		9.36		9.29		9.18		9.15		9.08		9.04	
L_r, ft		57.7		51.1		45.2		40.6		36.6		33.7	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.



W10

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W10×											
		49		45		39		33		30		26	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	5									126	188	107	161
	6							113	169	122	183	104	156
	7			141	212	125	187	111	166	104	157	89.3	134
	8	136	204	137	206	117	176	96.8	146	91.3	137	78.1	117
	9	134	201	122	183	104	156	86.1	129	81.2	122	69.4	104
	10	121	181	110	165	93.4	140	77.4	116	73.1	110	62.5	93.9
	11	110	165	99.6	150	84.9	128	70.4	106	66.4	99.8	56.8	85.4
	12	100	151	91.3	137	77.8	117	64.5	97.0	60.9	91.5	52.1	78.2
	13	92.7	139	84.3	127	71.9	108	59.6	89.5	56.2	84.5	48.1	72.2
	14	86.1	129	78.3	118	66.7	100	55.3	83.1	52.2	78.4	44.6	67.1
	15	80.4	121	73.1	110	62.3	93.6	51.6	77.6	48.7	73.2	41.7	62.6
	16	75.3	113	68.5	103	58.4	87.8	48.4	72.8	45.7	68.6	39.0	58.7
	17	70.9	107	64.5	96.9	54.9	82.6	45.6	68.5	43.0	64.6	36.8	55.2
	18	67.0	101	60.9	91.5	51.9	78.0	43.0	64.7	40.6	61.0	34.7	52.2
	19	63.5	95.4	57.7	86.7	49.2	73.9	40.8	61.3	38.4	57.8	32.9	49.4
	20	60.3	90.6	54.8	82.4	46.7	70.2	38.7	58.2	36.5	54.9	31.2	47.0
	21	57.4	86.3	52.2	78.4	44.5	66.9	36.9	55.4	34.8	52.3	29.8	44.7
	22	54.8	82.4	49.8	74.9	42.5	63.8	35.2	52.9	33.2	49.9	28.4	42.7
	23	52.4	78.8	47.6	71.6	40.6	61.0	33.7	50.6	31.8	47.7	27.2	40.8
	24	50.2	75.5	45.7	68.6	38.9	58.5	32.3	48.5	30.4	45.8	26.0	39.1
	25	48.2	72.5	43.8	65.9					29.2	43.9	25.0	37.6
	26									28.1	42.2		
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	1210	1810	1100	1650	934	1400	774	1160	731	1100	625	939
M_p/Ω_b	$\phi_b M_p$, kip-ft	151	227	137	206	117	176	96.8	146	91.3	137	78.1	117
M_r/Ω_b	$\phi_b M_r$, kip-ft	95.4	143	85.8	129	73.5	111	61.1	91.9	56.6	85.0	48.7	73.2
BF	BF, kips	2.44	3.67	2.59	3.89	2.51	3.77	2.39	3.59	3.08	4.62	2.90	4.36
V_n/Ω_v	$\phi_v V_n$, kips	68.0	102	70.7	106	62.5	93.7	56.4	84.7	62.8	94.2	53.7	80.6
Z_x, in.^{3}		60.4		54.9		46.8		38.8		36.6		31.3	
L_p, ft		8.97		7.10		6.99		6.85		4.84		4.80	
L_r, ft		31.6		26.9		24.2		21.8		16.1		14.9	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W10–W8

Shape		W10×										W8×	
		22		19		17		15		12^f		67	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	3					97.1	146	92.0	138	75.0	113		
	4			102	154	93.3	140	79.8	120	62.4	93.8		
	5	97.6	146	86.2	130	74.7	112	63.9	96.0	49.9	75.0		
	6	86.5	130	71.9	108	62.2	93.5	53.2	80.0	41.6	62.5	205	308
	7	74.1	111	61.6	92.6	53.3	80.1	45.6	68.6	35.7	53.6	200	300
	8	64.9	97.5	53.9	81.0	46.7	70.1	39.9	60.0	31.2	46.9	175	263
	9	57.7	86.7	47.9	72.0	41.5	62.3	35.5	53.3	27.7	41.7	155	234
	10	51.9	78.0	43.1	64.8	37.3	56.1	31.9	48.0	25.0	37.5	140	210
	11	47.2	70.9	39.2	58.9	33.9	51.0	29.0	43.6	22.7	34.1	127	191
	12	43.2	65.0	35.9	54.0	31.1	46.8	26.6	40.0	20.8	31.3	117	175
	13	39.9	60.0	33.2	49.8	28.7	43.2	24.6	36.9	19.2	28.9	108	162
	14	37.1	55.7	30.8	46.3	26.7	40.1	22.8	34.3	17.8	26.8	99.9	150
	15	34.6	52.0	28.7	43.2	24.9	37.4	21.3	32.0	16.6	25.0	93.3	140
	16	32.4	48.8	26.9	40.5	23.3	35.1	20.0	30.0	15.6	23.5	87.5	131
	17	30.5	45.9	25.4	38.1	22.0	33.0	18.8	28.2	14.7	22.1	82.3	124
	18	28.8	43.3	24.0	36.0	20.7	31.2	17.7	26.7	13.9	20.8	77.7	117
	19	27.3	41.1	22.7	34.1	19.6	29.5	16.8	25.3	13.1	19.7	73.6	111
	20	25.9	39.0	21.6	32.4	18.7	28.1	16.0	24.0	12.5	18.8	70.0	105
	21	24.7	37.1	20.5	30.9	17.8	26.7	15.2	22.9	11.9	17.9	66.6	100
	22	23.6	35.5	19.6	29.5	17.0	25.5	14.5	21.8	11.3	17.1	63.6	95.6
	23	22.6	33.9	18.7	28.2	16.2	24.4	13.9	20.9	10.9	16.3		
	24	21.6	32.5	18.0	27.0	15.6	23.4	13.3	20.0	10.4	15.6		
	25	20.8	31.2	17.2	25.9	14.9	22.4	12.8	19.2				

Beam Properties

W_c/Ω_b	$\phi_b W_c$, kip-ft	519	780	431	648	373	561	319	480	250	375	1400	2100
M_p/Ω_b	$\phi_b M_p$, kip-ft	64.9	97.5	53.9	81.0	46.7	70.1	39.9	60.0	31.2	46.9	175	263
M_r/Ω_b	$\phi_b M_r$, kip-ft	40.5	60.9	32.8	49.3	28.3	42.5	24.1	36.2	19.0	28.6	105	159
BF	BF, kips	2.68	4.02	3.17	4.77	2.99	4.49	2.75	4.14	2.35	3.53	1.73	2.60
V_n/Ω_v	$\phi_v V_n$, kips	48.8	73.2	51.2	76.8	48.5	72.8	46.0	69.0	37.5	56.3	103	154
Z_x, in.^{3}		26.0		21.6		18.7		16.0		12.6		70.1	
L_p, ft		4.70		3.09		2.98		2.86		2.87		7.49	
L_r, ft		13.8		9.72		9.13		8.61		8.05		47.7	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

W8

Table 3-6 (continued)
Maximum Total Uniform Load, kips
W Shapes

F_y = 50 ksi

Shape		W8×											
		58		48		40		35		31^f		28	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	5											**91.9**	138
	6	**179**	268			**119**	178	**101**	151	**91.2**	137	**90.5**	136
	7	**171**	256	**136**	204	**113**	171	**98.9**	149	**86.6**	130	**77.6**	117
	8	**149**	224	**122**	184	**99.3**	149	**86.6**	130	**75.8**	114	**67.9**	102
	9	**133**	199	**109**	163	**88.3**	133	**77.0**	116	**67.4**	101	**60.3**	90.7
	10	**119**	179	**97.8**	147	**79.4**	119	**69.3**	104	**60.6**	91.1	**54.3**	81.6
	11	**109**	163	**88.9**	134	**72.2**	109	**63.0**	94.6	**55.1**	82.8	**49.4**	74.2
	12	**99.5**	149	**81.5**	122	**66.2**	99.5	**57.7**	86.8	**50.5**	75.9	**45.2**	68.0
	13	**91.8**	138	**75.2**	113	**61.1**	91.8	**53.3**	80.1	**46.6**	70.1	**41.8**	62.8
	14	**85.3**	128	**69.9**	105	**56.7**	85.3	**49.5**	74.4	**43.3**	65.1	**38.8**	58.3
	15	**79.6**	120	**65.2**	98.0	**53.0**	79.6	**46.2**	69.4	**40.4**	60.7	**36.2**	54.4
	16	**74.6**	112	**61.1**	91.9	**49.7**	74.6	**43.3**	65.1	**37.9**	56.9	**33.9**	51.0
	17	**70.2**	106	**57.5**	86.5	**46.7**	70.2	**40.7**	61.2	**35.7**	53.6	**31.9**	48.0
	18	**66.3**	99.7	**54.3**	81.7	**44.1**	66.3	**38.5**	57.8	**33.7**	50.6	**30.2**	45.3
	19	**62.8**	94.4	**51.5**	77.4	**41.8**	62.8	**36.5**	54.8	**31.9**	48.0	**28.6**	42.9
	20	**59.7**	89.7	**48.9**	73.5	**39.7**	59.7	**34.6**	52.1	**30.3**	45.6	**27.1**	40.8
	21	**56.8**	85.4	**46.6**	70.0								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	**1190**	1790	**978**	1470	**794**	1190	**693**	1040	**606**	911	**543**	816
M_p/Ω_b	$\phi_b M_p$, kip-ft	**149**	224	**122**	184	**99.3**	149	**86.6**	130	**75.8**	114	**67.9**	102
M_r/Ω_b	$\phi_b M_r$, kip-ft	**90.8**	137	**75.4**	113	**62.0**	93.2	**54.5**	81.9	**48.0**	72.2	**42.4**	63.8
BF	BF, kips	**1.70**	2.56	**1.68**	2.53	**1.64**	2.47	**1.62**	2.43	**1.58**	2.37	**1.66**	2.50
V_n/Ω_v	$\phi_v V_n$, kips	**89.3**	134	**68.0**	102	**59.4**	89.1	**50.3**	75.5	**45.6**	68.4	**45.9**	68.9
Z_x, in.^{3}		59.8		49.0		39.8		34.7		30.4		27.2	
L_p, ft		7.42		7.35		7.21		7.17		7.18		5.72	
L_r, ft		41.7		35.2		29.9		27.0		24.8		21.0	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

F_y = 50 ksi

Table 3–6 (continued)
Maximum Total Uniform Load, kips
W Shapes

W8

Shape		W8× 24		21		18		15		13		10^f	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Span, ft	3							79.5	119	73.5	110	53.7	80.5
	4			82.8	124	74.9	112	67.9	102	56.9	85.5	43.7	65.7
	5	77.7	117	81.4	122	67.9	102	54.3	81.6	45.5	68.4	35.0	52.6
	6	76.8	115	67.9	102	56.6	85.0	45.2	68.0	37.9	57.0	29.2	43.8
	7	65.9	99.0	58.2	87.4	48.5	72.9	38.8	58.3	32.5	48.9	25.0	37.6
	8	57.6	86.6	50.9	76.5	42.4	63.8	33.9	51.0	28.4	42.8	21.9	32.9
	9	51.2	77.0	45.2	68.0	37.7	56.7	30.2	45.3	25.3	38.0	19.4	29.2
	10	46.1	69.3	40.7	61.2	33.9	51.0	27.1	40.8	22.8	34.2	17.5	26.3
	11	41.9	63.0	37.0	55.6	30.8	46.4	24.7	37.1	20.7	31.1	15.9	23.9
	12	38.4	57.7	33.9	51.0	28.3	42.5	22.6	34.0	19.0	28.5	14.6	21.9
	13	35.5	53.3	31.3	47.1	26.1	39.2	20.9	31.4	17.5	26.3	13.5	20.2
	14	32.9	49.5	29.1	43.7	24.2	36.4	19.4	29.1	16.3	24.4	12.5	18.8
	15	30.7	46.2	27.1	40.8	22.6	34.0	18.1	27.2	15.2	22.8	11.7	17.5
	16	28.8	43.3	25.4	38.2	21.2	31.9	17.0	25.5	14.2	21.4	10.9	16.4
	17	27.1	40.8	24.0	36.0	20.0	30.0	16.0	24.0	13.4	20.1	10.3	15.5
	18	25.6	38.5	22.6	34.0	18.9	28.3	15.1	22.7	12.6	19.0	9.72	14.6
	19	24.3	36.5	21.4	32.2	17.9	26.8	14.3	21.5	12.0	18.0	9.21	13.8
	20			20.4	30.6	17.0	25.5	13.6	20.4				

Beam Properties

		W8×24 ASD	LRFD	21 ASD	LRFD	18 ASD	LRFD	15 ASD	LRFD	13 ASD	LRFD	10 ASD	LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	**461**	693	**407**	612	**339**	510	**271**	408	**228**	342	**175**	263
M_p/Ω_b	$\phi_b M_p$, kip-ft	**57.6**	86.6	**50.9**	76.5	**42.4**	63.8	**33.9**	51.0	**28.4**	42.8	**21.9**	32.9
M_r/Ω_b	$\phi_b M_r$, kip-ft	**36.5**	54.9	**31.8**	47.8	**26.5**	39.9	**20.6**	31.0	**17.3**	26.0	**13.6**	20.5
BF	BF, kips	**1.59**	2.39	**1.86**	2.79	**1.74**	2.61	**1.92**	2.88	**1.76**	2.65	**1.52**	2.28
V_n/Ω_v	$\phi_v V_n$, kips	**38.9**	58.3	**41.4**	62.1	**37.4**	56.2	**39.7**	59.6	**36.8**	55.1	**26.8**	40.2
Z_x, in.^{3}		23.1		20.4		17.0		13.6		11.4		8.87	
L_p, ft		5.69		4.45		4.34		3.09		2.98		3.14	
L_r, ft		19.0		14.8		13.5		10.0		9.30		8.56	

ASD	**LRFD**
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For beams laterally unsupported, see Table 3–10.
Available strength tabulated above heavy line is limited by available shear strength.

S24–S20

Table 3–7
Maximum Total Uniform Load, kips
S Shapes

F_y = 36 ksi

Shape		S24×										S20×	
		121		106		100		90		80		96	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	6					515	772					468	702
	7	564	847			491	737	432	648			407	611
	8	550	826			429	645	399	599	346	518	356	535
	9	489	734	437	656	382	574	354	533	326	490	316	475
	10	440	661	401	603	343	516	319	480	293	441	285	428
	11	400	601	365	548	312	469	290	436	267	401	259	389
	12	366	551	334	502	286	430	266	400	244	367	237	356
	13	338	508	308	464	264	397	245	369	226	339	219	329
	14	314	472	286	430	245	369	228	343	209	315	203	305
	15	293	441	267	402	229	344	213	320	195	294	190	285
	16	275	413	251	377	215	323	199	300	183	275	178	267
	17	259	389	236	354	202	304	188	282	172	259	167	252
	18	244	367	223	335	191	287	177	266	163	245	158	238
	19	231	348	211	317	181	272	168	252	154	232	150	225
	20	220	330	200	301	172	258	160	240	147	220	142	214
	21	209	315	191	287	164	246	152	228	140	210	136	204
	22	200	300	182	274	156	235	145	218	133	200	129	194
	23	191	287	174	262	149	224	139	208	127	192	124	186
	24	183	275	167	251	143	215	133	200	122	184	119	178
	25	176	264	160	241	137	206	128	192	117	176	114	171
	26	169	254	154	232	132	199	123	184	113	169	109	164
	27	163	245	149	223	127	191	118	178	109	163	105	158
	28	157	236	143	215	123	184	114	171	105	157	102	153
	29	152	228	138	208	118	178	110	165	101	152	98.1	147
	30	147	220	134	201	114	172	106	160	97.7	147	94.9	143
	32	137	207	125	188	107	161	99.7	150	91.6	138	88.9	134
	34	129	194	118	177	101	152	93.8	141	86.2	130	83.7	126
	36	122	184	111	167	95.4	143	88.6	133	81.4	122	79.0	119
	38	116	174	106	159	90.4	136	84.0	126	77.2	116	74.9	113
	40	110	165	100	151	85.9	129	79.8	120	73.3	110	71.1	107
	42	105	157	95.5	143	81.8	123	76.0	114	69.8	105	67.8	102
	44	99.9	150	91.1	137	78.1	117	72.5	109	66.6	100	64.7	97.2
	46	95.6	144	87.2	131	74.7	112	69.4	104	63.7	95.8	61.9	93.0
	48	91.6	138	83.5	126	71.6	108	66.5	99.9	61.1	91.8	59.3	89.1
	50	88.0	132	80.2	121	68.7	103	63.8	95.9	58.6	88.1	56.9	85.5
	52	84.6	127	77.1	116	66.1	99.3	61.4	92.2	56.4	84.7		
	54	81.4	122	74.3	112	63.6	95.6	59.1	88.8	54.3	81.6		
	56	78.5	118	71.6	108	61.3	92.2	57.0	85.6	52.4	78.7		
	58	75.8	114	69.1	104	59.2	89.0	55.0	82.7	50.5	76.0		
	60	73.3	110	66.8	100	57.2	86.0	53.2	79.9	48.9	73.4		
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	4400	6610	4010	6030	3430	5160	3190	4800	2930	4410	2850	4280
M_p/Ω_b	$\phi_b M_p$, kip-ft	550	826	501	753	429	645	399	599	366	551	356	535
M_r/Ω_b	$\phi_b M_r$, kip-ft	324	488	302	454	250	376	235	353	220	331	207	312
BF	BF, kips	11.3	17.1	10.9	16.4	11.6	17.5	11.4	17.1	10.8	16.2	7.62	11.4
V_n/Ω_v	$\phi_v V_n$, kips	282	423	219	328	257	386	216	324	173	259	234	351
Z_x, in.^{3}		306		279		239		222		204		198	
L_p, ft		6.37		6.54		5.29		5.41		5.58		5.54	
L_r, ft		26.2		24.8		20.7		19.8		19.1		25.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3–7 (continued)
Maximum Total Uniform Load, kips
S Shapes

S20–S15

Shape		S20×						S18×				S15×	
		86		75		66		70		54.7		50	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	4							369	553			238	356
	5			366	549			356	536			221	333
	6	386	579	364	547	291	436	297	446	239	358	184	277
	7	376	565	312	469	285	429	255	383	214	321	158	238
	8	329	494	273	410	250	375	223	335	187	281	138	208
	9	292	439	243	365	222	334	198	298	166	250	123	185
	10	263	395	218	328	200	300	178	268	149	225	111	166
	11	239	359	199	298	182	273	162	243	136	204	101	151
	12	219	329	182	274	166	250	149	223	125	187	92.2	139
	13	202	304	168	253	154	231	137	206	115	173	85.1	128
	14	188	282	156	235	143	214	127	191	107	160	79.0	119
	15	175	264	146	219	133	200	119	179	99.6	150	73.8	111
	16	164	247	137	205	125	188	111	167	93.4	140	69.2	104
	17	155	233	128	193	118	177	105	158	87.9	132	65.1	97.8
	18	146	220	121	182	111	167	99.0	149	83.0	125	61.5	92.4
	19	138	208	115	173	105	158	93.8	141	78.7	118	58.2	87.5
	20	131	198	109	164	99.9	150	89.1	134	74.7	112	55.3	83.2
	21	125	188	104	156	95.1	143	84.9	128	71.2	107	52.7	79.2
	22	120	180	99.3	149	90.8	136	81.0	122	67.9	102	50.3	75.6
	23	114	172	95.0	143	86.9	131	77.5	116	65.0	97.7	48.1	72.3
	24	110	165	91.0	137	83.2	125	74.3	112	62.3	93.6	46.1	69.3
	25	105	158	87.4	131	79.9	120	71.3	107	59.8	89.9	44.3	66.5
	26	101	152	84.0	126	76.8	115	68.5	103	57.5	86.4	42.6	64.0
	27	97.4	146	80.9	122	74.0	111	66.0	99.2	55.4	83.2	41.0	61.6
	28	93.9	141	78.0	117	71.3	107	63.6	95.7	53.4	80.2	39.5	59.4
	29	90.7	136	75.3	113	68.9	104	61.4	92.4	51.5	77.5	38.2	57.4
	30	87.7	132	72.8	109	66.6	100	59.4	89.3	49.8	74.9	36.9	
	32	82.2	124	68.3	103	62.4	93.8	55.7	83.7	46.7	70.2	34.6	52.0
	34	77.4	116	64.2	96.6	58.8	88.3	52.4	78.8	44.0	66.1	32.5	48.9
	36	73.1	110	60.7	91.2	55.5	83.4	49.5	74.4	41.5	62.4	30.7	46.2
	38	69.2	104	57.5	86.4	52.6	79.0	46.9	70.5	39.3	59.1		
	40	65.7	98.8	54.6	82.1	49.9	75.1	44.6	67.0	37.4	56.2		
	42	62.6	94.1	52.0	78.2	47.6	71.5	42.4	63.8	35.6	53.5		
	44	59.8	89.8	49.6	74.6	45.4	68.2	40.5	60.9	34.0	51.1		
	46	57.2	85.9	47.5	71.4	43.4	65.3						
	48	54.8	82.4	45.5	68.4	41.6	62.6						
	50	52.6	79.1	43.7	65.7	40.0	60.0						

Beam Properties

W_c/Ω_b	$\phi_b W_c$, kip-ft	2630	3950	2180	3280	2000	3000	1780	2680	1490	2250	1110	1660
M_p/Ω_b	$\phi_b M_p$, kip-ft	329	494	273	410	250	375	223	335	187	281	138	208
M_r/Ω_b	$\phi_b M_r$, kip-ft	195	293	161	242	150	225	130	195	112	168	81.4	122
BF	BF, kips	7.55	11.3	7.76	11.7	7.50	11.3	6.11	9.19	5.99	9.00	4.09	6.14
V_n/Ω_v	$\phi_v V_n$, kips	193	289	183	274	145	218	184	276	119	179	119	178
Z_x, in.^{3}		183		152		139		124		104		77.0	
L_p, ft		5.66		4.83		4.95		4.50		4.75		4.29	
L_r, ft		23.4		19.3		18.3		19.7		17.3		18.2	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

S15–S10

Table 3–7 (continued)

Maximum Total Uniform Load, kips

S Shapes

F_y = 36 ksi

Shape		S15×		S12×								S10×	
		42.9		50		40.8		35		31.8		35	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2											171	257
	3			237	356							170	255
	4			219	329	160	240	148	222	121		127	191
	5	178	266	175	263	151	228	128	193	120	181	102	153
	6	166	249	146	219	126	190	107	161	100	150	84.8	127
	7		142	214	125	188	108	163	91.6	138	85.8	129	72.7
	8	124	187	109	164	94.7	142	80.1	120	75.1	113	63.6	95.6
	9	110	166	97.2	146	84.2	126	71.2	107	66.7	100	56.5	85.0
	10	99.4	149	87.5	132	75.7	114	64.1	96.3	60.1	90.3	50.9	76.5
	11	90.4	136	79.6	120	68.9	103	58.3	87.6	54.6	82.1	46.2	69.5
	12	82.9	125	72.9	110	63.1	94.9	53.4	80.3	50.1	75.2	42.4	63.7
	13	76.5	115	67.3	101	58.3	87.6	49.3	74.1	46.2	69.5	39.1	58.8
	14	71.0	107	62.5	94.0	54.1	81.3	45.8	68.8	42.9	64.5	36.3	54.6
	15	66.3	99.6	58.3	87.7	50.5	75.9	42.7	64.2	40.0	60.2	33.9	51.0
	16	62.2	93.4	54.7	82.2	47.3	71.1	40.1	60.2	37.5	56.4	31.8	47.8
	17	58.5	87.9	51.5	77.4	44.6	67.0	37.7	56.7	35.3	53.1	29.9	45.0
	18	55.2	83.0	48.6	73.1	42.1	63.2	35.6	53.5	33.4	50.2	28.3	42.5
	19	52.3	78.7	46.1	69.2	39.9	59.9	33.7	50.7	31.6	47.5	26.8	40.2
	20	49.7	74.7	43.8	65.8	37.9	56.9	32.0	48.2	30.0	45.1	25.4	38.2
	21	47.4	71.2	41.7	62.6	36.1	54.2	30.5	45.9	28.6	43.0	24.2	36.4
	22	45.2	67.9	39.8	59.8	34.4	51.7	29.1	43.8	27.3	41.0	23.1	34.8
	23	43.2	65.0	38.1	57.2	32.9	49.5	27.9	41.9	26.1	39.3	22.1	33.2
	24	41.4	62.3	36.5	54.8	31.6	47.4	26.7	40.1	25.0	37.6	21.2	31.9
	25	39.8	59.8	35.0	52.6	30.3	45.5	25.6	38.5	24.0	36.1	20.3	30.6
	26	38.2	57.5	33.7	50.6	29.1	43.8	24.7	37.1	23.1	34.7		
	27	36.8	55.4	32.4	48.7	28.1	42.2	23.7	35.7	22.2	33.4		
	28	35.5	53.4	31.3	47.0	27.0	40.7	22.9	34.4	21.5	32.2		
	29	34.3	51.5	30.2	45.4	26.1	39.3	22.1	33.2	20.7	31.1		
	30	33.1	49.8	29.2	43.8	25.2	37.9	21.4	32.1	20.0	30.1		
	32	31.1	46.7										
	34	29.2	44.0										
	36	27.6	41.5										
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	994	1490	875	1320	757	1140	641	963	601	903	509	765
M_p/Ω_b	$\phi_b M_p$, kip-ft	124	187	109	164	94.7	142	80.1	120	75.1	113	63.6	95.6
M_r/Ω_b	$\phi_b M_r$, kip-ft	74.7	112	63.6	95.6	56.7	85.2	47.9	72.0	45.5	68.4	37.0	55.6
BF	BF, kips	3.99	6.00	2.22	3.34	2.31	3.48	2.46	3.69	2.42	3.63	1.51	2.27
V_n/Ω_v	$\phi_v V_n$, kips	88.8	133	119	178	79.8	120	74.0	111	60.5	90.7	85.5	128
Z_x, in.^{3}		69.2		60.9		52.7		44.6		41.8		35.4	
L_p, ft		4.41		4.29		4.41		4.08		4.16		3.74	
L_r, ft		16.8		24.9		20.8		17.2		16.4		21.4	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3–7 (continued)
Maximum Total Uniform Load, kips
S Shapes

S10–S5

Shape		S10×		S8×				S6×				S5×	
		25.4		23		18.4		17.2		12.5		10	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2			102	152			75.4	113			30.8	46.2
	3			92.0	138	62.4	93.7	50.3	75.6	40.1	60.1	27.1	40.8
	4	89.6	134	69.0	104	59.3	89.1	37.7	56.7	30.4	45.6	20.3	30.6
	5	81.3	122	55.2	82.9	47.4	71.3	30.2	45.4	24.3	36.5	16.3	24.5
	6	67.8	102	46.0	69.1	39.5	59.4	25.1	37.8	20.2	30.4	13.6	20.4
	7	109	58.1	87.3	39.4	59.2	33.9	50.9	21.6	32.4	17.3	26.1	11.6
	8	50.8	76.4	34.5	51.8	29.6	44.6	18.9	28.4	15.2	22.8	10.2	15.3
	9	45.2	67.9	30.7	46.1	26.3	39.6	16.8	25.2	13.5	20.3	9.04	13.6
	10	40.7	61.1	27.6	41.5	23.7	35.6	15.1	22.7	12.1	18.3	8.13	12.2
	11	37.0	55.6	25.1	37.7	21.6	32.4	13.7	20.6	11.0	16.6	7.39	11.1
	12	33.9	50.9	23.0	34.6	19.8	29.7	12.6	18.9	10.1	15.2	6.78	10.2
	13	31.3	47.0	21.2	31.9	18.2	27.4	11.6	17.4	9.34	14.0		
	14	29.1	43.7	19.7	29.6	16.9	25.5	10.8	16.2	8.67	13.0		
	15	27.1	40.8	18.4	27.6	15.8	23.8	10.1	15.1	8.10	12.2		
	16	25.4	38.2	17.2	25.9	14.8	22.3						
	17	23.9	36.0	16.2	24.4	13.9	21.0						
	18	22.6	34.0	15.3	23.0	13.2	19.8						
	19	21.4	32.2	14.5	21.8	12.5	18.8						
	20	20.3	30.6	13.8	20.7	11.9	17.8						
	21	19.4	29.1										
	22	18.5	27.8										
	23	17.7	26.6										
	24	16.9	25.5										
	25	16.3	24.5										
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	407	611	276	415	237	356	151	227	121	183	81.3	122
M_p/Ω_b	$\phi_b M_p$, kip-ft	50.8	76.4	34.5	51.8	29.6	44.6	18.9	28.4	15.2	22.8	10.2	15.3
M_r/Ω_b	$\phi_b M_r$, kip-ft	30.9	46.5	20.4	30.6	18.1	27.2	11.0	16.5	9.23	13.9	6.16	9.26
BF	BF, kips	1.58	2.38	0.948	1.42	0.974	1.46	0.459	0.691	0.515	0.775	0.341	0.512
V_n/Ω_v	$\phi_v V_n$, kips	44.8	67.2	50.8	76.2	31.2	46.8	40.2	60.3	20.0	30.1	15.4	23.1
Z_x, in.^{3}		28.3		19.2		16.5		10.5		8.45		5.66	
L_p, ft		3.95		3.31		3.44		2.80		2.92		2.66	
L_r, ft		16.5		18.2		15.3		19.9		14.5		14.4	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

S4–S3

Table 3–7 (continued)
Maximum Total Uniform Load, kips
S Shapes

F_y = 50 ksi

Shape		S4×				S3×			
		9.5		7.7		7.5		5.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	**29.0**	43.6	**22.2**	33.4	**16.9**	25.4	**13.9**	21.0
	3	**19.4**	29.1	**16.8**	25.2	**11.3**	16.9	**9.29**	14.0
	4	**14.5**	21.8	**12.6**	18.9	**8.44**	12.7	**6.97**	10.5
	5	**11.6**	17.5	**10.1**	15.1	**6.75**	10.2	**5.58**	8.38
	6	**9.68**	14.5	**8.38**	12.6	**5.63**	8.46	**4.65**	6.98
	7	**17.5**	8.29	**12.5**	7.19	**4.82**	7.25	**3.98**	5.99
	8	**7.26**	10.9	**6.29**	9.45				
	9	**6.45**	9.70	**5.59**	8.40				
	10	**5.81**	8.73	**5.03**	7.56				

Beam Properties

		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W_c/Ω_b	$\phi_b W_c$, kip-ft	**58.1**	87.3	**50.3**	75.6	**33.8**	50.8	**27.9**	41.9
M_p/Ω_b	$\phi_b M_p$, kip-ft	**7.26**	10.9	**6.29**	9.45	**4.22**	6.35	**3.49**	5.24
M_r/Ω_b	$\phi_b M_r$, kip-ft	**4.25**	6.39	**3.81**	5.73	**2.44**	3.67	**2.10**	3.16
BF	BF, kips	**0.190**	0.286	**0.202**	0.304	**0.0899**	0.135	**0.102**	0.154
V_n/Ω_v	$\phi_v V_n$, kips	**18.8**	28.2	**11.1**	16.7	**15.1**	22.6	**7.34**	11.0
Z_x, in.^{3}		4.04		3.50		2.35		1.94	
L_p, ft		2.35		2.40		2.14		2.16	
L_r, ft		18.2		14.7		22.0		15.7	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.50$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–7 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3–8
Maximum Total Uniform Load, kips
C Shapes

C15–C12

Shape		C15×						C12×					
		50		40		33.9		30		25		20.7	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	3	278	418					158	238	120	181		
	4	246	370	202	303	155	233	121	182	106	159	87.5	132
	5	197	296	165	248	146	219	97.1	146	84.4	127	73.5	111
	6	164	247	138	207	122	183	80.9	122	70.3	106	61.3	92.1
	7	141	211	118	177	104	157	69.4	104	60.3	90.6	52.5	79.0
	8	123	185	103	155	91.2	137	60.7	91.2	52.8	79.3	46.0	69.1
	9	109	164	91.8	138	81.0	122	54.0	81.1	46.9	70.5	40.9	61.4
	10	98.5	148	82.6	124	72.9	110	48.6	73.0	42.2	63.4	36.8	55.3
	11	89.5	135	75.1	113	66.3	99.7	44.2	66.4	38.4	57.7	33.4	50.2
	12	82.1	123	68.9	104	60.8	91.4	40.5	60.8	35.2	52.9	30.6	46.1
	13	75.8	114	63.6	95.5	56.1	84.3	37.4	56.2	32.5	48.8	28.3	42.5
	14	70.3	106	59.0	88.7	52.1	78.3	34.7	52.1	30.1	45.3	26.3	39.5
	15	65.7	98.7	55.1	82.8	48.6	73.1	32.4	48.7	28.1	42.3	24.5	36.8
	16	61.6	92.5	51.7	77.6	45.6	68.5	30.4	45.6	26.4	39.6	23.0	34.5
	17	57.9	87.1	48.6	73.1	42.9	64.5	28.6	42.9	24.8	37.3	21.6	32.5
	18	54.7	82.2	45.9	69.0	40.5	60.9	27.0	40.6	23.4	35.2	20.4	30.7
	19	51.8	77.9	43.5	65.4	38.4	57.7	25.6	38.4	22.2	33.4	19.4	29.1
	20	49.2	74.0	41.3	62.1	36.5	54.8	24.3	36.5	21.1	31.7	18.4	27.6
	21	46.9	70.5	39.4	59.1	34.7	52.2	23.1	34.8	20.1	30.2	17.5	26.3
	22	44.8	67.3	37.6	56.5	33.2	49.8	22.1	33.2	19.2	28.8	16.7	25.1
	23	42.8	64.4	35.9	54.0	31.7	47.7	21.1	31.7	18.4	27.6	16.0	24.0
	24	41.0	61.7	34.4	51.8	30.4	45.7	20.2	30.4	17.6	26.4	15.3	23.0
	25	39.4	59.2	33.1	49.7	29.2	43.9	19.4	29.2	16.9	25.4	14.7	22.1
	26	37.9	56.9	31.8	47.8	28.1	42.2	18.7	28.1	16.2	24.4	14.1	21.3
	27	36.5	54.8	30.6	46.0	27.0	40.6	18.0	27.0	15.6	23.5	13.6	20.5
	28	35.2	52.9	29.5	44.4	26.1	39.2	17.3	26.1	15.1	22.7	13.1	19.7
	29	34.0	51.0	28.5	42.8	25.2	37.8	16.7	25.2	14.6	21.9	12.7	19.1
	30	32.8	49.3	27.5	41.4	24.3	36.5	16.2	24.3	14.1	21.1	12.3	18.4
	31	31.8	47.8	26.7	40.1	23.5	35.4						
	32	30.8	46.3	25.8	38.8	22.8	34.3						
	33	29.8	44.9	25.0	37.6	22.1	33.2						
	34	29.0	43.5	24.3	36.5	21.5	32.2						
	35	28.1	42.3	23.6	35.5	20.8	31.3						
	36	27.4	41.1	23.0	34.5	20.3	30.5						
	37	26.6	40.0	22.3	33.6	19.7	29.6						
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	985	1480	826	1240	729	1100	486	730	422	634	368	553
M_p/Ω_b	$\phi_b M_p$, kip-ft	123	185	103	155	91.2	137	60.7	91.2	52.8	79.3	46.0	69.1
M_r/Ω_b	$\phi_b M_r$, kip-ft	67.7	102	58.4	87.8	52.8	79.3	33.9	51.0	30.2	45.4	27.1	40.7
BF	BF, kips	3.48	5.23	3.63	5.45	3.57	5.37	2.18	3.27	2.22	3.33	2.15	3.23
V_n/Ω_v	$\phi_v V_n$, kips	139	209	101	152	77.6	117	79.2	119	60.1	90.3	43.8	65.8
Z_x, in.^{3}		68.5		57.5		50.8		33.8		29.4		25.6	
L_p, ft		3.60		3.68		3.75		3.17		3.24		3.32	
L_r, ft		19.5		16.0		14.5		15.5		13.4		12.1	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.67$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–8 is used.
Available strength tabulated above heavy line is limited by available shear strength.

C10–C9

Table 3–8 (continued)
Maximum Total Uniform Load, kips
S Shapes

F_y = 36 ksi

Shape		C10×								C9×	
		30		25		20		15.3		20	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	174	262	136	205	98.0	147			104	157
	3	128	193	111	166	92.9	140	62.1	93.3	81.2	122
	4	96.1	144	82.9	125	69.7	105	57.2	86.0	60.9	91.5
	5	76.9	116	66.3	99.6	55.7	83.8	45.7	68.8	48.7	73.2
	6	64.1	96.3	55.3	83.0	46.4	69.8	38.1	57.3	40.6	61.0
	7	54.9	82.5	47.4	71.2	39.8	59.8	32.7	49.1	34.8	52.3
	8	48.0	72.2	41.4	62.3	34.8	52.4	28.6	43.0	30.4	45.7
	9	42.7	64.2	36.8	55.4	31.0	46.5	25.4	38.2	27.1	40.7
	10	38.4	57.8	33.2	49.8	27.9	41.9	22.9	34.4	24.3	36.6
	11	34.9	52.5	30.1	45.3	25.3	38.1	20.8	31.3	22.1	33.3
	12	32.0	48.1	27.6	41.5	23.2	34.9	19.1	28.7	20.3	30.5
	13	29.6	44.4	25.5	38.3	21.4	32.2	17.6	26.4	18.7	28.1
	14	27.5	41.3	23.7	35.6	19.9	29.9	16.3	24.6	17.4	26.1
	15	25.6	38.5	22.1	33.2	18.6	27.9	15.2	22.9	16.2	24.4
	16	24.0	36.1	20.7	31.1	17.4	26.2	14.3	21.5	15.2	22.9
	17	22.6	34.0	19.5	29.3	16.4	24.6	13.5	20.2	14.3	21.5
	18	21.4	32.1	18.4	27.7	15.5	23.3	12.7	19.1	13.5	20.3
	19	20.2	30.4	17.4	26.2	14.7	22.0	12.0	18.1	12.8	19.3
	20	19.2	28.9	16.6	24.9	13.9	20.9	11.4	17.2	12.2	18.3
	21	18.3	27.5	15.8	23.7	13.3	19.9	10.9	16.4	11.6	17.4
	22	17.5	26.3	15.1	22.6	12.7	19.0	10.4	15.6	11.1	16.6
	23	16.7	25.1	14.4	21.7	12.1	18.2	9.95	14.9		
	24	16.0	24.1	13.8	20.8	11.6	17.5	9.53	14.3		
	25	15.4	23.1	13.3	19.9	11.1	16.8	9.15	13.8		

Beam Properties

W_c/Ω_b	$\phi_b W_c$, kip-ft	384	578	332	498	279	419	229	344	243	366
M_p/Ω_b	$\phi_b M_p$, kip-ft	48.0	72.2	41.4	62.3	34.8	52.4	28.6	43.0	30.4	45.7
M_r/Ω_b	$\phi_b M_r$, kip-ft	26.0	39.1	22.9	34.4	19.8	29.8	16.9	25.4	17.0	25.6
BF	BF, kips	1.27	1.91	1.40	2.10	1.48	2.23	1.45	2.18	1.13	1.70
V_n/Ω_v	$\phi_v V_n$, kips	87.0	131	68.0	102	49.0	73.7	31.0	46.7	52.2	78.4

Z_x, in.^{3}	26.7	23.1	19.4	15.9	16.9
L_p, ft	2.78	2.81	2.87	2.96	2.66
L_r, ft	20.1	16.1	13.0	11.0	14.5

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.67$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–8 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3–8 (continued)
Maximum Total Uniform Load, kips
C Shapes

C9–C8

Shape		C9×				C8×					
		15		13.4		18.5		13.7		11.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	66.4	99.7			99.9	150	62.7	94.2		
	3	65.3	98.2	54.2	81.5	66.6	100	52.5	78.9	45.5	68.4
	4	49.0	73.7	45.2	68.0	49.9	75.1	39.4	59.2	34.6	52.0
	5	39.2	58.9	36.2	54.4	40.0	60.0	31.5	47.3	27.7	41.6
	6	32.7	49.1	30.2	45.3	33.3	50.0	26.2	39.4	23.1	34.7
	7	28.0	42.1	25.8	38.8	28.5	42.9	22.5	33.8	19.8	29.7
	8	24.5	36.8	22.6	34.0	25.0	37.5	19.7	29.6	17.3	26.0
	9	21.8	32.7	20.1	30.2	22.2	33.4	17.5	26.3	15.4	23.1
	10	19.6	29.5	18.1	27.2	20.0	30.0	15.7	23.7	13.8	20.8
	11	17.8	26.8	16.4	24.7	18.2	27.3	14.3	21.5	12.6	18.9
	12	16.3	24.6	15.1	22.7	16.6	25.0	13.1	19.7	11.5	17.3
	13	15.1	22.7	13.9	20.9	15.4	23.1	12.1	18.2	10.6	16.0
	14	14.0	21.0	12.9	19.4	14.3	21.4	11.2	16.9	9.88	14.9
	15	13.1	19.6	12.1	18.1	13.3	20.0	10.5	15.8	9.22	13.9
	16	12.3	18.4	11.3	17.0	12.5	18.8	9.84	14.8	8.65	13.0
	17	11.5	17.3	10.6	16.0	11.8	17.7	9.26	13.9	8.14	12.2
	18	10.9	16.4	10.1	15.1	11.1	16.7	8.75	13.1	7.69	11.6
	19	10.3	15.5	9.52	14.3	10.5	15.8	8.29	12.5	7.28	10.9
	20	9.80	14.7	9.05	13.6	9.99	15.0	7.87	11.8	6.92	10.4
	21	9.34	14.0	8.61	12.9						
	22	8.91	13.4	8.22	12.4						
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	196	295	181	272	200	300	157	237	138	208
M_p/Ω_b	$\phi_b M_p$, kip-ft	24.5	36.8	22.6	34.0	25.0	37.5	19.7	29.6	17.3	26.0
M_r/Ω_b	$\phi_b M_r$, kip-ft	14.2	21.4	13.4	20.1	13.8	20.8	11.3	17.1	10.2	15.4
BF	BF, kips	1.18	1.78	1.16	1.75	0.822	1.24	0.909	1.37	0.903	1.36
V_n/Ω_v	$\phi_v V_n$, kips	33.2	49.9	27.1	40.8	50.4	75.7	31.4	47.1	22.8	34.2
Z_x, in.^{3}		13.6		12.6		13.9		11.0		9.63	
L_p, ft		2.74		2.77		2.49		2.55		2.59	
L_r, ft		11.4		10.7		16.1		11.7		10.4	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.67$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–8 is used.
Available strength tabulated above heavy line is limited by available shear strength.

C7–C6

Table 3–8 (continued)
Maximum Total Uniform Load, kips
C Shapes

F_y = 36 ksi

Shape		C7×						C6×			
		14.7		12.2		9.8		13		10.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	70.0	105	56.9	85.5	38.0	57.2	52.4	78.7	44.4	66.8
	3	46.7	70.2	40.5	60.9	34.4	51.7	34.9	52.5	29.6	44.5
	4	35.0	52.6	30.4	45.7	25.8	38.8	26.2	39.4	22.2	33.4
	5	28.0	42.1	24.3	36.5	20.7	31.0	21.0	31.5	17.8	26.7
	6	23.3	35.1	20.3	30.5	17.2	25.9	17.5	26.2	14.8	22.3
	7	20.0	30.1	17.4	26.1	14.8	22.2	15.0	22.5	12.7	19.1
	8	17.5	26.3	15.2	22.8	12.9	19.4	13.1	19.7	11.1	16.7
	9	15.6	23.4	13.5	20.3	11.5	17.2	11.6	17.5	9.87	14.8
	10	14.0	21.1	12.2	18.3	10.3	15.5	10.5	15.7	8.89	13.4
	11	12.7	19.1	11.1	16.6	9.39	14.1	9.52	14.3	8.08	12.1
	12	11.7	17.5	10.1	15.2	8.61	12.9	8.73	13.1	7.40	11.1
	13	10.8	16.2	9.35	14.1	7.94	11.9	8.06	12.1	6.83	10.3
	14	10.0	15.0	8.68	13.1	7.38	11.1	7.48	11.2	6.35	9.54
	15	9.34	14.0	8.11	12.2	6.88	10.3	6.98	10.5	5.92	8.90
	16	8.75	13.2	7.60	11.4	6.45	9.70				
	17	8.24	12.4	7.15	10.7	6.07	9.13				
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	140	211	122	183	103	155	105	157	88.9	134
M_p/Ω_b	$\phi_b M_p$, kip-ft	17.5	26.3	15.2	22.8	12.9	19.4	13.1	19.7	11.1	16.7
M_r/Ω_b	$\phi_b M_r$, kip-ft	9.78	14.7	8.70	13.1	7.63	11.5	7.26	10.9	6.34	9.52
BF	BF, kips	0.620	0.932	0.664	0.998	0.674	1.01	0.413	0.620	0.458	0.688
V_n/Ω_v	$\phi_v V_n$, kips	37.9	57.0	28.4	42.7	19.0	28.6	33.9	51.0	24.4	36.6
Z_x, in.^{3}		9.75		8.46		7.19		7.29		6.18	
L_p, ft		2.34		2.37		2.40		2.18		2.20	
L_r, ft		14.8		12.1		10.2		16.3		12.6	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–8 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3–8 (continued)
Maximum Total Uniform Load, kips
C Shapes

C6–C4

Shape		C6×		C5×				C4×			
		8.2		9		6.7		7.2		5.4	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	31.0	46.7	31.6	47.4	24.6	36.9	20.4	30.7	16.5	24.8
	3	24.7	37.1	21.0	31.6	17.0	25.6	13.6	20.5	11.0	16.5
	4	18.5	27.8	15.8	23.7	12.8	19.2	10.2	15.3	8.24	12.4
	5	14.8	22.3	12.6	19.0	10.2	15.3	8.17	12.3	6.59	9.91
	6	12.4	18.6	10.5	15.8	8.50	12.8	6.81	10.2	5.49	8.26
	7	10.6	15.9	9.02	13.6	7.29	11.0	5.83	8.77	4.71	7.08
	8	9.26	13.9	7.89	11.9	6.38	9.58	5.10	7.67	4.12	6.19
	9	8.23	12.4	7.02	10.5	5.67	8.52	4.54	6.82	3.66	5.50
	10	7.41	11.1	6.31	9.49	5.10	7.67	4.08	6.14	3.30	4.95
	11	6.74	10.1	5.74	8.63	4.64	6.97				
	12	6.18	9.28	5.26	7.91	4.25	6.39				
	13	5.70	8.57								
	14	5.29	7.96								
	15	4.94	7.43								
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	74.1	111	63.1	94.9	51.0	76.7	40.8	61.4	33.0	49.5
M_p/Ω_b	$\phi_b M_p$, kip-ft	9.26	13.9	7.89	11.9	6.38	9.58	5.10	7.67	4.12	6.19
M_r/Ω_b	$\phi_b M_r$, kip-ft	5.48	8.23	4.47	6.72	3.76	5.66	2.88	4.33	2.42	3.64
BF	BF, kips	0.477	0.717	0.289	0.435	0.314	0.471	0.165	0.249	0.185	0.279
V_n/Ω_v	$\phi_v V_n$, kips	15.5	23.3	21.0	31.6	12.3	18.5	16.6	25.0	9.52	14.3
Z_x, in.^{3}		5.16		4.39		3.55		2.84		2.29	
L_p, ft		2.23		2.02		2.04		1.86		1.85	
L_r, ft		10.2		13.9		10.4		15.3		11.0	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–8 is used.
Available strength tabulated above heavy line is limited by available shear strength.

C4–C3

Table 3–8 (continued)
Maximum Total Uniform Load, kips
C Shapes

F_y = 36 ksi

Shape		C4×		C3×							
		4.5		6		5		4.1		3.5	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	12.9	19.4	12.5	18.8	10.9	16.4	9.50	14.3	8.88	13.3
	3	10.2	15.3	8.34	12.5	7.28	10.9	6.33	9.52	5.92	8.90
	4	7.62	11.5	6.25	9.40	5.46	8.21	4.75	7.14	4.44	6.67
	5	6.10	9.16	5.00	7.52	4.37	6.56	3.80	5.71	3.55	5.34
	6	5.08	7.64	4.17	6.26	3.64	5.47	3.17	4.76	2.96	4.45
	7	4.35	6.54	3.57	5.37	3.12	4.69	2.71	4.08	2.54	3.81
	8	3.81	5.73								
	9	3.39	5.09								
	10	3.05	4.58								
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	30.5	45.8	25.0	37.6	21.8	32.8	19.0	28.5	17.8	26.7
M_p/Ω_b	$\phi_b M_p$, kip-ft	3.81	5.73	3.13	4.70	2.73	4.10	2.37	3.57	2.22	3.34
M_r/Ω_b	$\phi_b M_r$, kip-ft	2.30	3.45	1.74	2.61	1.55	2.33	1.39	2.08	1.31	1.97
BF	BF, kips	0.185	0.278	0.0760	0.114	0.0860	0.129	0.0933	0.140	0.0952	0.143
V_n/Ω_v	$\phi_v V_n$, kips	6.47	9.72	13.8	20.8	10.0	15.0	6.60	9.91	5.12	7.70
Z_x, in.^{3}		2.12		1.74		1.52		1.32		1.24	
L_p, ft		1.90		1.72		1.69		1.66		1.64	
L_r, ft		10.1		20.0		15.4		12.3		11.2	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–8 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3–9
Maximum Total Uniform Load, kips
MC Shapes

MC18–MC13

Shape		MC18×								MC13×			
		58		51.9		45.8		42.7		50		40	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	3									265	398	188	283
	4	326	490	279	420	233	350			219	328	184	277
	5	274	412	251	377	228	342	210	315	175	263	147	221
	6	228	343	209	314	190	285	180	270	146	219	123	184
	7	196	294	179	269	163	244	154	232	125	188	105	158
	8	171	257	157	236	142	214	135	203	109	164	92.0	138
	9	152	229	139	209	126	190	120	180	97.1	146	81.8	123
	10	137	206	125	188	114	171	108	162	87.4	131	73.6	111
	11	125	187	114	171	103	155	98.1	147	79.5	119	66.9	101
	12	114	172	105	157	94.8	142	89.9	135	72.8	109	61.4	92.2
	13	105	158	96.5	145	87.5	132	83.0	125	67.2	101	56.6	85.1
	14	97.9	147	89.6	135	81.3	122	77.1	116	62.4	93.8	52.6	79.0
	15	91.4	137	83.6	126	75.8	114	72.0	108	58.3	87.6	49.1	73.8
	16	85.7	129	78.4	118	71.1	107	67.5	101	54.6	82.1	46.0	69.2
	17	80.6	121	73.8	111	66.9	101	63.5	95.4	51.4	77.3	43.3	65.1
	18	76.1	114	69.7	105	63.2	95.0	60.0	90.1	48.6	73.0	40.9	61.5
	19	72.1	108	66.0	99.2	59.9	90.0	56.8	85.4	46.0	69.1	38.7	58.2
	20	68.5	103	62.7	94.2	56.9	85.5	54.0	81.1	43.7	65.7	36.8	55.3
	21	65.3	98.1	59.7	89.8	54.2	81.4	51.4	77.3	41.6	62.6	35.1	52.7
	22	62.3	93.6	57.0	85.7	51.7	77.7	49.1	73.7	39.7	59.7	33.5	50.3
	23	59.6	89.6	54.5	81.9	49.5	74.3	46.9	70.5	38.0	57.1	32.0	48.1
	24	57.1	85.8	52.3	78.5	47.4	71.2	45.0	67.6	36.4	54.7	30.7	46.1
	25	54.8	82.4	50.2	75.4	45.5	68.4	43.2	64.9	35.0	52.5	29.4	44.3
	26	52.7	79.2	48.2	72.5	43.8	65.8	41.5	62.4	33.6	50.5	28.3	42.6
	27	50.8	76.3	46.4	69.8	42.1	63.3	40.0	60.1	32.4	48.7	27.3	41.0
	28	48.9	73.6	44.8	67.3	40.6	61.1	38.5	57.9	31.2	46.9	26.3	39.5
	29	47.3	71.0	43.2	65.0	39.2	59.0	37.2	55.9	30.1	45.3	25.4	38.2
	30	45.7	68.7	41.8	62.8	37.9	57.0	36.0	54.1	29.1	43.8	24.5	36.9
	32	42.8	64.4	39.2	58.9	35.5	53.4	33.7	50.7	27.3	41.1	23.0	34.6
	34	40.3	60.6	36.9	55.4	33.5	50.3	31.7	47.7				
	36	38.1	57.2	34.8	52.4	31.6	47.5	30.0	45.1				
	38	36.1	54.2	33.0	49.6	29.9	45.0	28.4	42.7				
	40	34.3	51.5	31.4	47.1	28.4	42.7	27.0	40.6				
	42	32.6	49.0	29.9	44.9	27.1	40.7	25.7	38.6				
	44	31.1	46.8	28.5	42.8	25.9	38.9	24.5	36.9				
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	1370	2060	1250	1880	1140	1710	1080	1620	874	1310	736	1110
M_p/Ω_b	$\phi_b M_p$, kip-ft	171	257	157	236	142	214	135	203	109	164	92.0	138
M_r/Ω_b	$\phi_b M_r$, kip-ft	94.4	142	87.6	132	80.8	121	77.4	116	60.8	91.4	52.7	79.3
BF	BF, kips	5.18	7.78	5.25	7.89	5.21	7.84	5.14	7.73	2.09	3.15	2.29	3.44
V_n/Ω_v	$\phi_v V_n$, kips	163	245	140	210	116	175	105	157	132	199	94.2	142
Z_x, in.^{3}		95.4		87.3		79.2		75.1		60.8		51.2	
L_p, ft		4.23		4.30		4.39		4.44		4.40		4.49	
L_r, ft		19.1		17.5		16.2		15.6		27.6		21.7	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–9 is used.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3–9 (continued)
Maximum Total Uniform Load, kips

MC Shapes

MC13–MC12

F_y = 36 ksi

Shape		MC13×				MC12×							
		35		31.8		50		45		40		35	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	3					259	390	220	331	183	275		
	4	150	226	126	190	203	305	187	281	171	257	144	217
	5	134	201	125	188	162	244	149	225	137	206	124	186
	6	111	167	104	156	135	203	124	187	114	172	103	155
	7	95.4	143	89.1	134	116	174	107	160	97.8	147	88.6	133
	8	83.4	125	78.0	117	101	152	93.4	140	85.6	129	77.5	117
	9	74.2	111	69.3	104	90.2	136	83.0	125	76.1	114	68.9	104
	10	66.8	100	62.4	93.8	81.2	122	74.7	112	68.5	103	62.0	93.2
	11	60.7	91.2	56.7	85.2	73.8	111	67.9	102	62.3	93.6	56.4	84.7
	12	55.6	83.6	52.0	78.1	67.6	102	62.2	93.6	57.1	85.8	51.7	77.7
	13	51.4	77.2	48.0	72.1	62.4	93.8	57.5	86.4	52.7	79.2	47.7	71.7
	14	47.7	71.7	44.6	67.0	58.0	87.1	53.4	80.2	48.9	73.5	44.3	66.6
	15	44.5	66.9	41.6	62.5	54.1	81.3	49.8	74.8	45.7	68.6	41.3	62.1
	16	41.7	62.7	39.0	58.6	50.7	76.2	46.7	70.2	42.8	64.3	38.8	58.3
	17	39.3	59.0	36.7	55.2	47.7	71.8	43.9	66.0	40.3	60.6	36.5	54.8
	18	37.1	55.7	34.7	52.1	45.1	67.8	41.5	62.4	38.0	57.2	34.5	51.8
	19	35.1	52.8	32.8	49.4	42.7	64.2	39.3	59.1	36.0	54.2	32.6	49.1
	20	33.4	50.2	31.2	46.9	40.6	61.0	37.3	56.1	34.2	51.5	31.0	46.6
	21	31.8	47.8	29.7	44.7	38.7	58.1	35.6	53.5	32.6	49.0	29.5	44.4
	22	30.3	45.6	28.4	42.6	36.9	55.5	34.0	51.0	31.1	46.8	28.2	42.4
	23	29.0	43.6	27.1	40.8	35.3	53.0	32.5	48.8	29.8	44.8	27.0	40.5
	24	27.8	41.8	26.0	39.1	33.8	50.8	31.1	46.8	28.5	42.9	25.8	38.8
	25	26.7	40.1	25.0	37.5	32.5	48.8	29.9	44.9	27.4	41.2	24.8	37.3
	26	25.7	38.6	24.0	36.1	31.2	46.9	28.7	43.2	26.3	39.6	23.9	35.9
	27	24.7	37.2	23.1	34.7	30.1	45.2	27.7	41.6	25.4	38.1	23.0	34.5
	28	23.8	35.8	22.3	33.5	29.0	43.6	26.7	40.1	24.5	36.8	22.2	33.3
	29	23.0	34.6	21.5	32.3	28.0	42.1	25.8	38.7	23.6	35.5	21.4	32.1
	30	22.3	33.4	20.8	31.3	27.1	40.7	24.9	37.4	22.8	34.3	20.7	31.1
	32	20.9	31.4	19.5	29.3								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	668	1000	624	938	812	1220	747	1120	685	1030	620	932
M_p/Ω_b	$\phi_b M_p$, kip-ft	83.4	125	78.0	117	101	152	93.4	140	85.6	129	77.5	117
M_r/Ω_b	$\phi_b M_r$, kip-ft	48.7	73.3	46.2	69.4	56.5	84.9	52.7	79.2	49.1	73.7	45.3	68.1
BF	BF, kips	2.33	3.50	2.32	3.48	1.66	2.50	1.77	2.66	1.86	2.80	1.92	2.88
V_n/Ω_v	$\phi_v V_n$, kips	75.2	113	63.1	94.8	130	195	110	166	91.6	138	72.2	108
Z_x, in.^{3}		46.5		43.4		56.5		52.0		47.7		43.2	
L_p, ft		4.55		4.59		4.53		4.55		4.58		4.61	
L_r, ft		19.5		18.3		31.6		27.5		24.2		21.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–9 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3–9 (continued)
Maximum Total Uniform Load, kips
MC Shapes

MC12–MC10

Shape		MC12×				MC10×							
		31		10.6		41.1		33.6		28.5		25	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2			59.0	88.6	206	309						
	3			55.7	83.7	188	283	149	224	110	165	98.3	148
	4	115	173	41.8	62.8	141	212	121	182	108	162	94.0	141
	5	114	172	33.4	50.2	113	170	97.0	146	86.2	130	75.2	113
	6	95.2	143	27.9	41.9	94.1	141	80.8	121	71.8	108	62.7	94.2
	7	81.6	123	23.9	35.9	80.6	121	69.3	104	61.6	92.6	53.7	80.7
	8	71.4	107	20.9	31.4	70.5	106	60.6	91.1	53.9	81.0	47.0	70.6
	9	63.5	95.4	18.6	27.9	62.7	94.2	53.9	81.0	47.9	72.0	41.8	62.8
	10	57.1	85.8	16.7	25.1	56.4	84.8	48.5	72.9	43.1	64.8	37.6	56.5
	11	51.9	78.0	15.2	22.8	51.3	77.1	44.1	66.3	39.2	58.9	34.2	51.4
	12	47.6	71.5	13.9	20.9	47.0	70.7	40.4	60.7	35.9	54.0	31.3	47.1
	13	43.9	66.0	12.9	19.3	43.4	65.2	37.3	56.1	33.2	49.8	28.9	43.5
	14	40.8	61.3	11.9	17.9	40.3	60.6	34.6	52.1	30.8	46.3	26.9	40.4
	15	38.1	57.2	11.1	16.7	37.6	56.5	32.3	48.6	28.7	43.2	25.1	37.7
	16	35.7	53.6	10.4	15.7	35.3	53.0	30.3	45.6	26.9	40.5	23.5	35.3
	17	33.6	50.5	9.83	14.8	33.2	49.9	28.5	42.9	25.4	38.1	22.1	33.2
	18	31.7	47.7	9.29	14.0	31.4	47.1	26.9	40.5	23.9	36.0	20.9	31.4
	19	30.1	45.2	8.80	13.2	29.7	44.6	25.5	38.4	22.7	34.1	19.8	29.7
	20	28.6	42.9	8.36	12.6	28.2	42.4	24.2	36.4	21.6	32.4	18.8	28.3
	21	27.2	40.9	7.96	12.0	26.9	40.4	23.1	34.7	20.5	30.9	17.9	26.9
	22	26.0	39.0	7.60	11.4	25.7	38.6	22.0	33.1	19.6	29.4	17.1	25.7
	23	24.8	37.3	7.27	10.9	24.5	36.9	21.1	31.7	18.7	28.2	16.3	24.6
	24	23.8	35.8	6.96	10.5	23.5	35.3	20.2	30.4	18.0	27.0	15.7	23.5
	25	22.8	34.3	6.69	10.0	22.6	33.9	19.4	29.2	17.2	25.9	15.0	22.6
	26	22.0	33.0	6.43	9.66								
	27	21.2	31.8	6.19	9.31								
	28	20.4	30.7	5.97	8.97								
	29	19.7	29.6	5.76	8.66								
	30	19.0	28.6	5.57	8.37								
Beam Properties													
W_c/Ω_b	$\phi_b W_c$, kip-ft	571	858	167	251	564	848	485	729	431	648	376	565
M_p/Ω_b	$\phi_b M_p$, kip-ft	71.4	107	20.9	31.4	70.5	106	60.6	91.1	53.9	81.0	47.0	70.6
M_r/Ω_b	$\phi_b M_r$, kip-ft	42.4	63.8	11.6	17.4	39.6	59.5	34.9	52.5	31.8	47.8	27.7	41.6
BF	BF, kips	1.91	2.88	2.76	4.16	1.00	1.50	1.14	1.71	1.22	1.83	1.29	1.94
V_n/Ω_v	$\phi_v V_n$, kips	57.4	86.3	29.5	44.3	103	155	74.4	112	55.0	82.6	49.1	73.9
Z_x, in.^{3}		39.7		11.6		39.3		33.7		30.0		26.2	
L_p, ft		4.63		1.45		4.74		4.80		4.84		4.13	
L_r, ft		19.8		4.82		35.7		27.3		23.0		19.1	

ASD	LRFD
$\Omega_b = 1.67$ $\Omega_v = 1.67$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–9 is used.
Available strength tabulated above heavy line is limited by available shear strength.

Table 3–9 (continued)

Maximum Total Uniform Load, kips

MC Shapes

MC10–MC8

F_y = 36 ksi

Shape		**MC10×**						**MC9×**				**MC8×**	
		22		**8.4**		**6.5**		**25.4**		**23.9**		**22.8**	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Span, ft	**2**			**44.0**	66.1	**39.3**	59.1						
	3			**37.9**	57.0	**28.3**	42.5	**105**	157	**93.1**	140	**88.4**	133
	4	**75.0**	113	**28.4**	42.8	**21.2**	31.9	**84.4**	127	**80.8**	121	**68.5**	103
	5	**68.7**	103	**22.8**	34.2	**17.0**	25.5	**67.5**	102	**64.6**	97.1	**54.8**	82.3
	6	**57.3**	86.1	**19.0**	28.5	**14.1**	21.2	**56.3**	84.6	**53.9**	81.0	**45.6**	68.6
	7	**49.1**	73.8	**16.3**	24.4	**12.1**	18.2	**48.2**	72.5	**46.2**	69.4	**39.1**	58.8
	8	**43.0**	64.6	**14.2**	21.4	**10.6**	15.9	**42.2**	63.5	**40.4**	60.7	**34.2**	51.4
	9	**38.2**	57.4	**12.6**	19.0	**9.42**	14.2	**37.5**	56.4	**35.9**	54.0	**30.4**	45.7
	10	**34.4**	51.7	**11.4**	17.1	**8.48**	12.7	**33.8**	50.8	**32.3**	48.6	**27.4**	41.2
	11	**31.2**	47.0	**10.3**	15.5	**7.71**	11.6	**30.7**	46.1	**29.4**	44.2	**24.9**	37.4
	12	**28.6**	43.0	**9.48**	14.3	**7.06**	10.6	**28.1**	42.3	**26.9**	40.5	**22.8**	34.3
	13	**26.4**	39.7	**8.75**	13.2	**6.52**	9.80	**26.0**	39.0	**24.9**	37.4	**21.1**	31.7
	14	**24.5**	36.9	**8.13**	12.2	**6.06**	9.10	**24.1**	36.3	**23.1**	34.7	**19.6**	29.4
	15	**22.9**	34.4	**7.59**	11.4	**5.65**	8.49	**22.5**	33.8	**21.5**	32.4	**18.3**	27.4
	16	**21.5**	32.3	**7.11**	10.7	**5.30**	7.96	**21.1**	31.7	**20.2**	30.4	**17.1**	25.7
	17	**20.2**	30.4	**6.69**	10.1	**4.99**	7.49	**19.9**	29.9	**19.0**	28.6	**16.1**	24.2
	18	**19.1**	28.7	**6.32**	9.50	**4.71**	7.08	**18.8**	28.2	**18.0**	27.0	**15.2**	22.9
	19	**18.1**	27.2	**5.99**	9.00	**4.46**	6.71	**17.8**	26.7	**17.0**	25.6	**14.4**	21.7
	20	**17.2**	25.8	**5.69**	8.55	**4.24**	6.37	**16.9**	25.4	**16.2**	24.3	**13.7**	20.6
	21	**16.4**	24.6	**5.42**	8.14	**4.04**	6.07	**16.1**	24.2	**15.4**	23.1		
	22	**15.6**	23.5	**5.17**	7.77	**3.85**	5.79	**15.4**	23.1	**14.7**	22.1		
	23	**14.9**	22.5	**4.95**	7.44	**3.69**	5.54						
	24	**14.3**	21.5	**4.74**	7.13	**3.53**	5.31						
	25	**13.7**	20.7	**4.55**	6.84	**3.39**	5.10						

Beam Properties

W_c/Ω_b	$\phi_b W_c$, kip-ft	**344**	517	**114**	171	**84.8**	127	**338**	508	**323**	486	**274**	412
M_p/Ω_b	$\phi_b M_p$, kip-ft	**43.0**	64.6	**14.2**	21.4	**10.6**	15.9	**42.2**	63.5	**40.4**	60.7	**34.2**	51.4
M_r/Ω_b	$\phi_b M_r$, kip-ft	**25.8**	38.7	**8.03**	12.1	**5.77**	8.67	**24.6**	36.9	**23.7**	35.7	**20.0**	30.1
BF	BF, kips	**1.29**	1.93	**1.76**	2.65	**1.94**	2.92	**0.966**	1.45	**0.983**	1.48	**0.720**	1.08
V_n/Ω_v	$\phi_v V_n$, kips	**37.5**	56.4	**22.0**	33.0	**19.7**	29.5	**52.4**	78.7	**46.6**	70.0	**44.2**	66.4

Z_x, in.^{3}	23.9	7.92	5.90	23.5	22.5	19.1
L_p, ft	4.15	1.52	1.09	4.19	4.20	4.26
L_r, ft	17.5	5.03	3.58	22.5	21.2	24.0

ASD	**LRFD**
$\Omega_b = 1.67$ $\Omega_v = 1.67$	$\phi_b = 0.90$ $\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–9 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

Table 3–9 (continued)
Maximum Total Uniform Load, kips
MC Shapes

MC8–MC7

Shape		MC8×								MC7×			
		21.4		20		18.7		8.5		22.7		19.1	
Design		**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Span, ft	**2**			**82.8**	124			**37.0**	55.7	**91.1**	137		
	3	**77.6**	117	**78.5**	118	**73.1**	110	**33.3**	50.0	**78.4**	118	**63.7**	95.8
	4	**65.5**	98.4	**58.9**	88.5	**56.2**	84.5	**25.0**	37.5	**58.8**	88.3	**52.1**	78.4
	5	**52.4**	78.7	**47.1**	70.8	**45.0**	67.6	**20.0**	30.0	**47.0**	70.7	**41.7**	62.7
	6	**43.6**	65.6	**39.3**	59.0	**37.5**	56.3	**16.6**	25.0	**39.2**	58.9	**34.8**	52.2
	7	**37.4**	56.2	**33.7**	50.6	**32.1**	48.3	**14.3**	21.4	**33.6**	50.5	**29.8**	44.8
	8	**32.7**	49.2	**29.5**	44.3	**28.1**	42.2	**12.5**	18.8	**29.4**	44.2	**26.1**	39.2
	9	**29.1**	43.7	**26.2**	39.3	**25.0**	37.5	**11.1**	16.7	**26.1**	39.3	**23.2**	34.8
	10	**26.2**	39.4	**23.6**	35.4	**22.5**	33.8	**9.98**	15.0	**23.5**	35.3	**20.9**	31.3
	11	**23.8**	35.8	**21.4**	32.2	**20.4**	30.7	**9.07**	13.6	**21.4**	32.1	**19.0**	28.5
	12	**21.8**	32.8	**19.6**	29.5	**18.7**	28.2	**8.32**	12.5	**19.6**	29.4	**17.4**	26.1
	13	**20.1**	30.3	**18.1**	27.2	**17.3**	26.0	**7.68**	11.5	**18.1**	27.2	**16.0**	24.1
	14	**18.7**	28.1	**16.8**	25.3	**16.1**	24.1	**7.13**	10.7	**16.8**	25.2	**14.9**	22.4
	15	**17.5**	26.2	**15.7**	23.6	**15.0**	22.5	**6.65**	10.0	**15.7**	23.6	**13.9**	20.9
	16	**16.4**	24.6	**14.7**	22.1	**14.1**	21.1	**6.24**	9.38	**14.7**	22.1	**13.0**	19.6
	17	**15.4**	23.2	**13.9**	20.8	**13.2**	19.9	**5.87**	8.83	**13.8**	20.8	**12.3**	18.4
	18	**14.5**	21.9	**13.1**	19.7	**12.5**	18.8	**5.55**	8.33				
	19	**13.8**	20.7	**12.4**	18.6	**11.8**	17.8	**5.25**	7.90				
	20	**13.1**	19.7	**11.8**	17.7	**11.2**	16.9	**4.99**	7.50				

Beam Properties

W_c/Ω_b	$\phi_b W_c$, kip-ft	**262**	394	**236**	354	**225**	338	**99.8**	150	**235**	353	**209**	313
M_p/Ω_b	$\phi_b M_p$, kip-ft	**32.7**	49.2	**29.5**	44.3	**28.1**	42.2	**12.5**	18.8	**29.4**	44.2	**26.1**	39.2
M_r/Ω_b	$\phi_b M_r$, kip-ft	**19.3**	29.1	**17.1**	25.7	**16.5**	24.8	**7.32**	11.0	**17.0**	25.6	**15.5**	23.3
BF	BF, kips	**0.735**	1.10	**0.769**	1.16	**0.783**	1.18	**0.967**	1.45	**0.488**	0.734	**0.530**	0.797
V_n/Ω_v	$\phi_v V_n$, kips	**38.8**	58.3	**41.4**	62.2	**36.5**	54.9	**18.5**	27.8	**45.5**	68.4	**31.9**	47.9
Z_x, in.^{3}		18.2		16.4		15.6		6.95		16.4		14.5	
L_p, ft		4.26		3.61		3.62		2.08		4.34		4.32	
L_r, ft		22.5		19.7		18.5		7.42		29.6		24.3	

ASD	LRFD	
$\Omega_b = 1.67$ $\Omega_v = 1.67$	$\phi_b = 0.90$ $\phi_v = 1.00$	Note: Beams must be laterally supported if Table 3–9 is used. Available strength tabulated above heavy line is limited by available shear strength.

MC6

Table 3–9 (continued)
Maximum Total Uniform Load, kips
MC Shapes

F_y = 36 ksi

Shape		MC6×									
		18		15.3		16.3		15.1		12	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	58.8	88.4	52.8	79.3	58.2	87.5	49.0	73.7	48.1	72.3
	3	56.1	84.3	47.5	71.4	49.6	74.6	47.1	70.8	35.8	53.8
	4	42.1	63.2	35.6	53.5	37.2	56.0	35.3	53.1	26.8	40.3
	5	33.7	50.6	28.5	42.8	29.8	44.8	28.3	42.5	21.5	32.3
	6	28.0	42.2	23.7	35.7	24.8	37.3	23.5	35.4	17.9	26.9
	7	24.0	36.1	20.3	30.6	21.3	32.0	20.2	30.3	15.3	23.1
	8	21.0	31.6	17.8	26.8	18.6	28.0	17.7	26.5	13.4	20.2
	9	18.7	28.1	15.8	23.8	16.5	24.9	15.7	23.6	11.9	17.9
	10	16.8	25.3	14.2	21.4	14.9	22.4	14.1	21.2	10.7	16.1
	11	15.3	23.0	12.9	19.5	13.5	20.3	12.8	19.3	9.76	14.7
	12	14.0	21.1	11.9	17.8	12.4	18.7	11.8	17.7	8.95	13.4
	13	12.9	19.5	11.0	16.5	11.5	17.2	10.9	16.3	8.26	12.4
	14	12.0	18.1	10.2	15.3	10.6	16.0	10.1	15.2	7.67	11.5
	15	11.2	16.9	9.49	14.3	9.93	14.9	9.42	14.2	7.16	10.8
Beam Properties											
W_c/Ω_b	$\phi_b W_c$, kip-ft	168	253	142	214	149	224	141	212	107	161
M_p/Ω_b	$\phi_b M_p$, kip-ft	21.0	31.6	17.8	26.8	18.6	28.0	17.7	26.5	13.4	20.2
M_r/Ω_b	$\phi_b M_r$, kip-ft	12.4	18.7	10.6	16.0	10.9	16.4	10.4	15.7	7.84	11.8
BF	BF, kips	0.356	0.536	0.372	0.559	0.371	0.557	0.381	0.572	0.416	0.625
V_n/Ω_v	$\phi_v V_n$, kips	29.4	44.2	26.4	39.7	29.1	43.7	24.5	36.9	24.1	36.2
Z_x, in.^{3}		11.7		9.91		10.4		9.83		7.47	
L_p, ft		4.39		4.36		3.69		3.68		3.02	
L_r, ft		28.5		23.7		24.5		22.6		16.4	

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$
$\Omega_v = 1.67$	$\phi_v = 1.00$

Note: Beams must be laterally supported if Table 3–9 is used.
Available strength tabulated above heavy line is limited by available shear strength.

F_y = 36 ksi

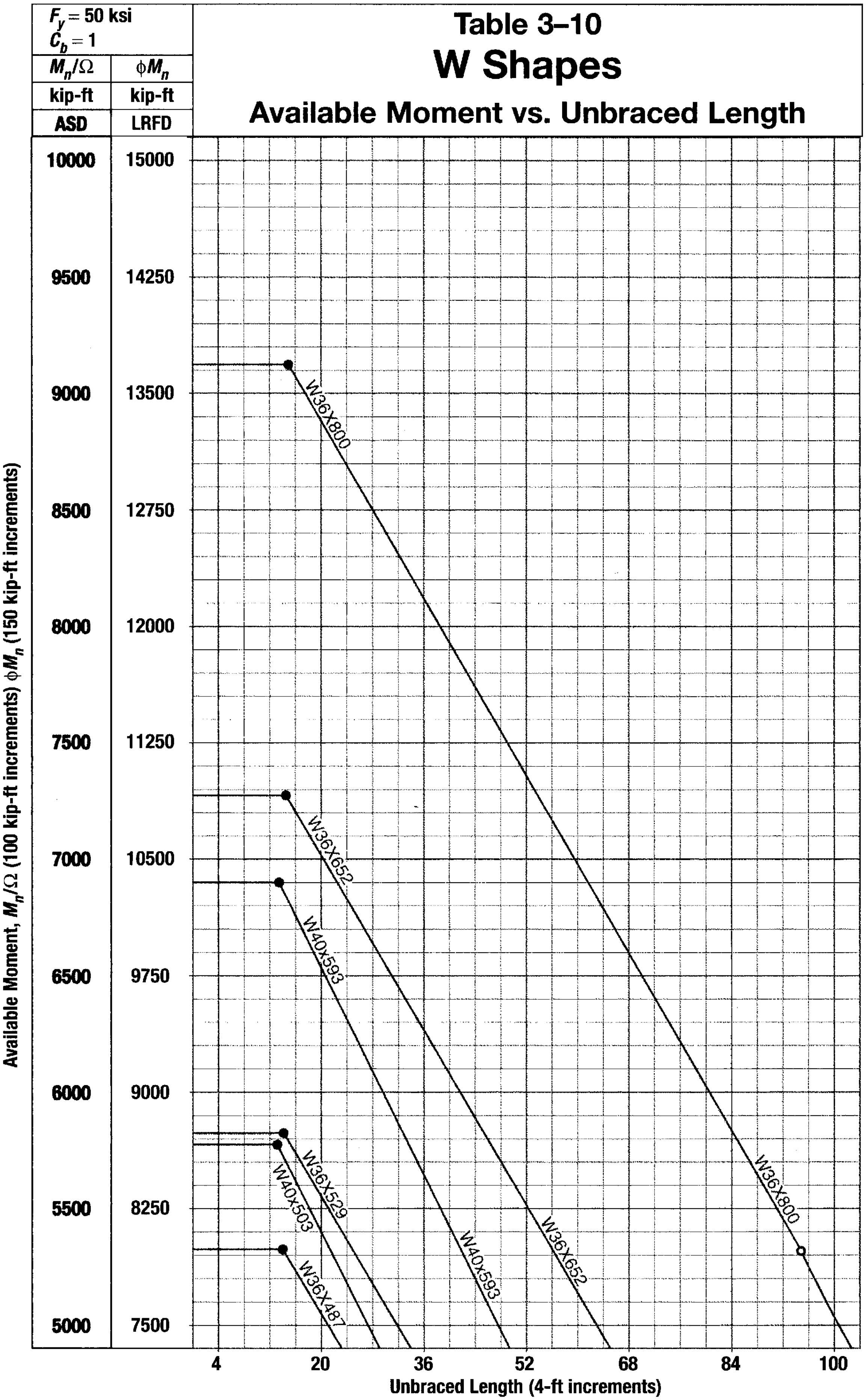
Table 3–9 (continued)
Maximum Total Uniform Load, kips
MC Shapes

MC6–MC3

Shape		MC6×				MC4×		MC3×	
		7		6.5		13.8		7.1	
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Span, ft	2	27.8	41.8	24.1	36.2	39.8	59.8	16.1	24.2
	3	21.6	32.4	20.5	30.8	26.5	39.9	10.7	16.1
	4	16.2	24.3	15.4	23.1	19.9	29.9	8.04	12.1
	5	12.9	19.4	12.3	18.5	15.9	23.9	6.44	9.67
	6	10.8	16.2	10.3	15.4	13.3	19.9	5.36	8.06
	7	9.24	13.9	8.79	13.2	11.4	17.1	4.60	6.91
	8	8.09	12.2	7.69	11.6	9.94	14.9		
	9	7.19	10.8	6.84	10.3	8.84	13.3		
	10	6.47	9.72	6.16	9.25	7.95	12.0		
	11	5.88	8.84	5.60	8.41				
	12	5.39	8.10	5.13	7.71				
	13	4.98	7.48	4.73	7.12				
	14	4.62	6.94	4.40	6.61				
	15	4.31	6.48	4.10	6.17				
Beam Properties									
W_c/Ω_b	$\phi_b W_c$, kip-ft	64.7	97.2	61.6	92.5	79.5	120	32.2	48.4
M_p/Ω_b	$\phi_b M_p$, kip-ft	8.09	12.2	7.69	11.6	9.94	14.9	4.02	6.05
M_r/Ω_b	$\phi_b M_r$, kip-ft	4.79	7.20	4.60	6.92	5.57	8.36	2.28	3.42
BF	BF, kips	0.490	0.736	0.486	0.730	0.126	0.190	0.0747	0.112
V_n/Ω_v	$\phi_v V_n$, kips	13.9	20.9	12.0	18.1	25.9	38.9	12.1	18.2
Z_x, in.^{3}		4.50		4.28		5.53		2.24	
L_p, ft		2.23		2.24		3.03		2.34	
L_r, ft		8.97		8.60		37.7		25.7	

ASD	LRFD	
$\Omega_b = 1.67$ $\Omega_v = 1.67$	$\phi_b = 0.90$ $\phi_v = 1.00$	Note: Beams must be laterally supported if Table 3–9 is used. Available strength tabulated above heavy line is limited by available shear strength.

Table 3–10
W Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi $C_b = 1$	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

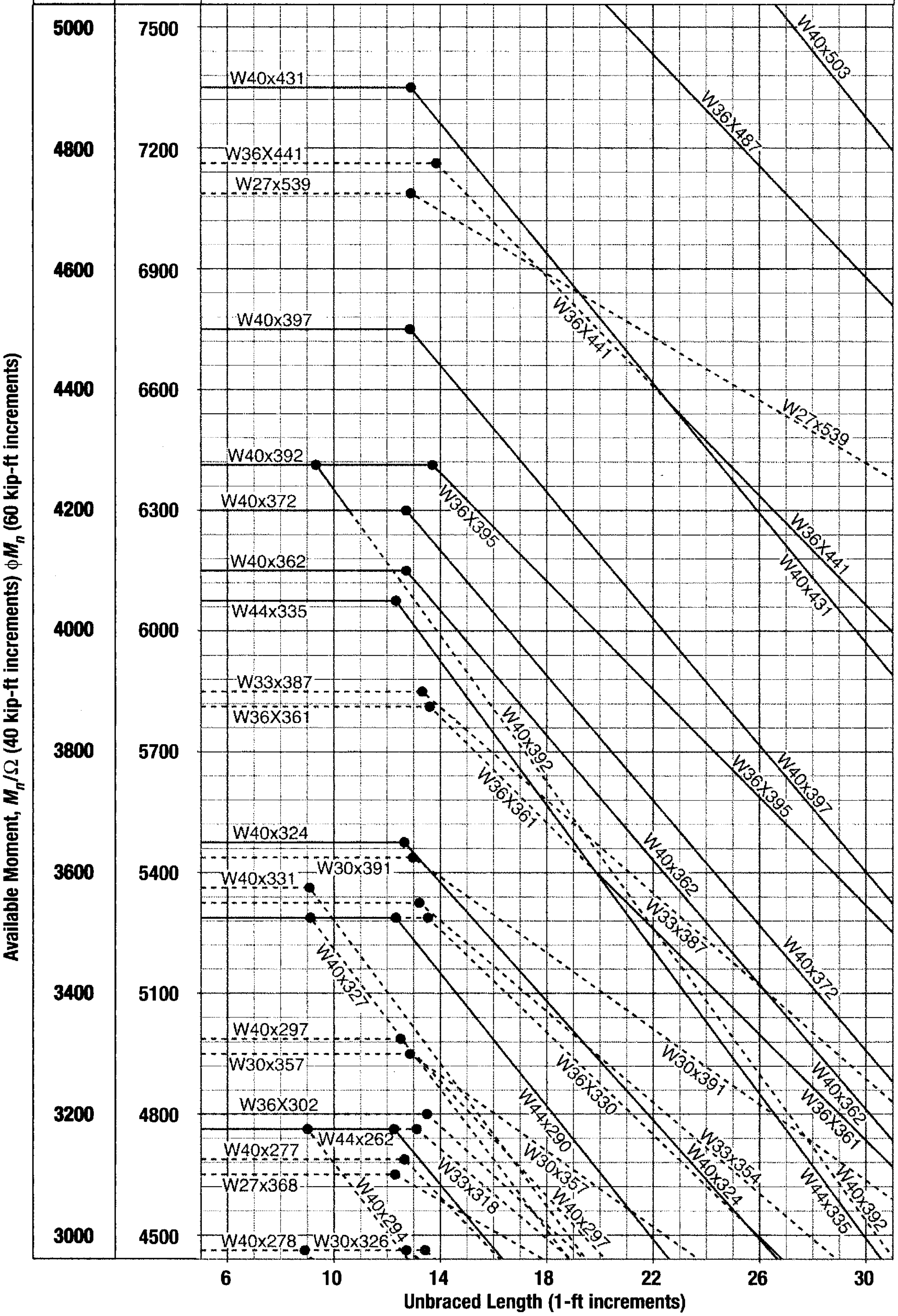


Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length

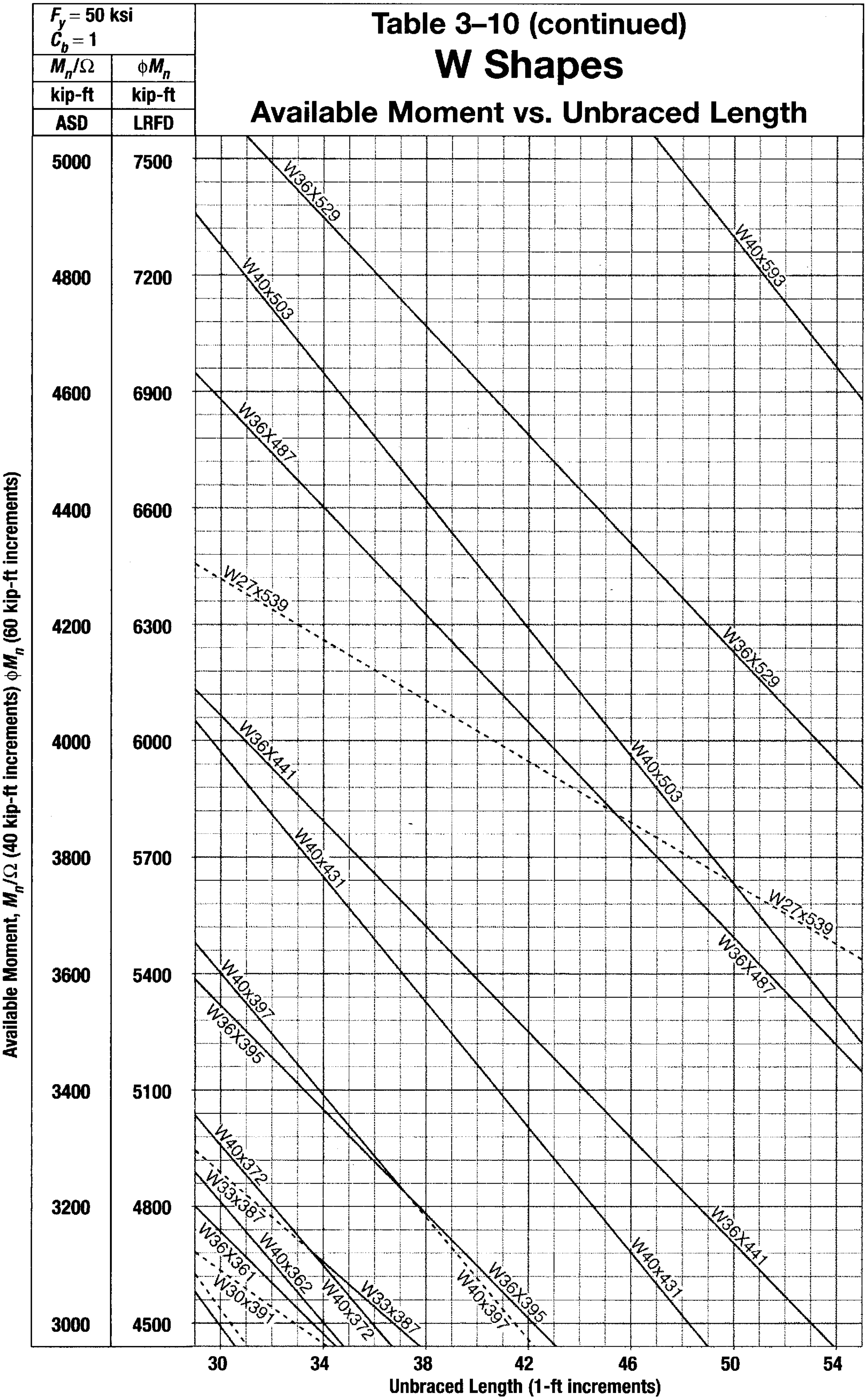
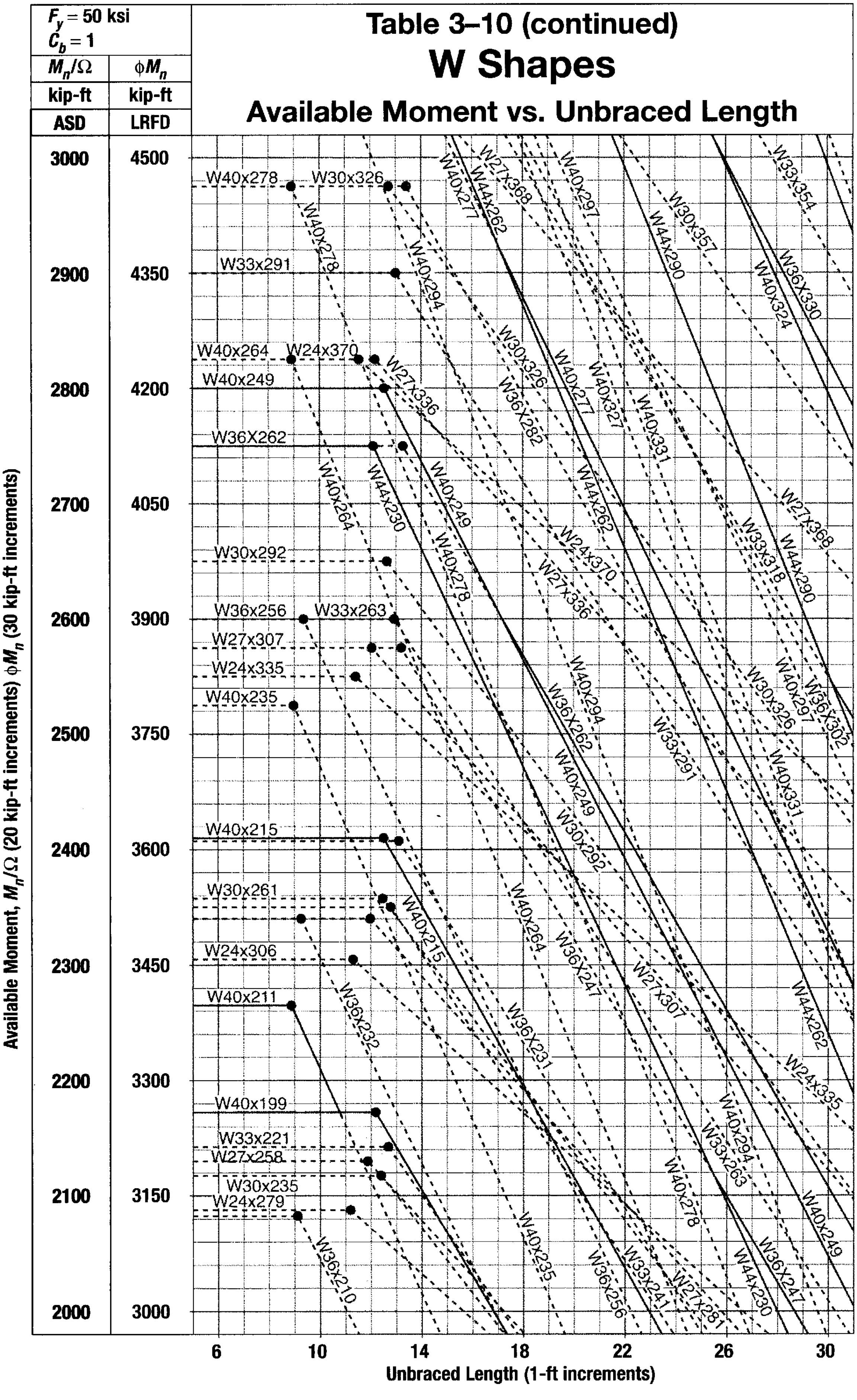


Table 3–10 (continued)

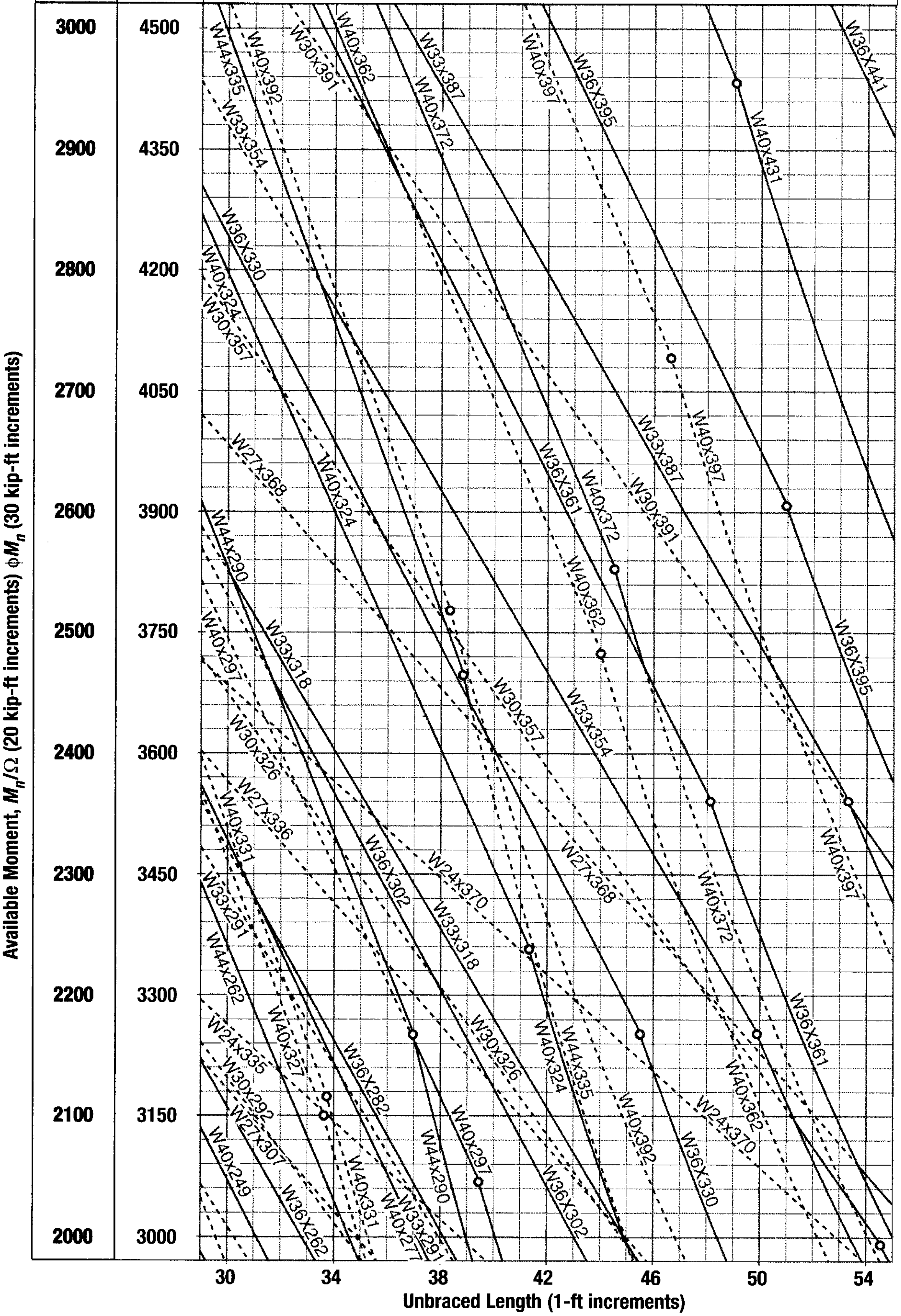
W Shapes

Available Moment vs. Unbraced Length



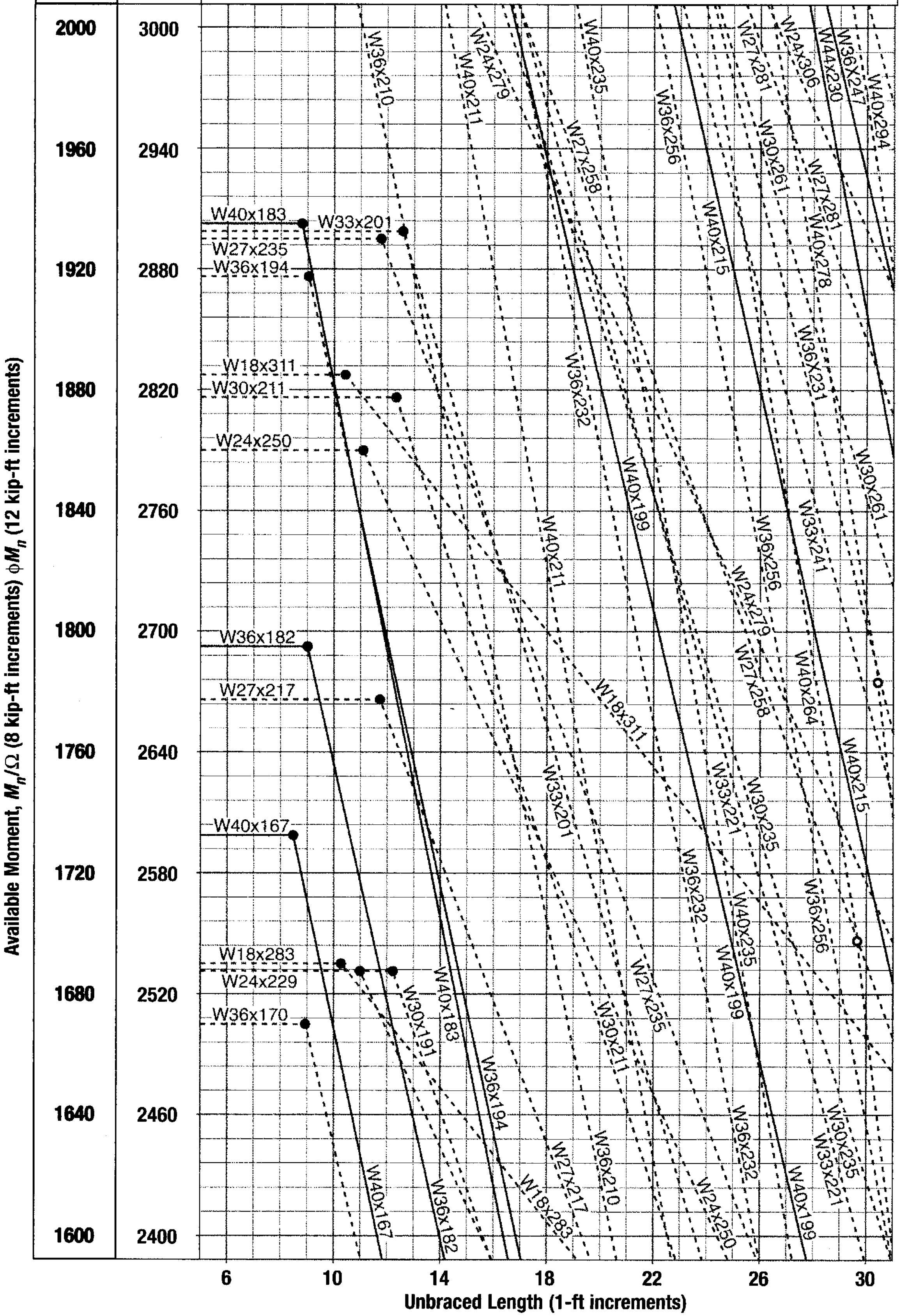
$F_y = 50$ ksi $C_b = 1$	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi C_b = 1	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

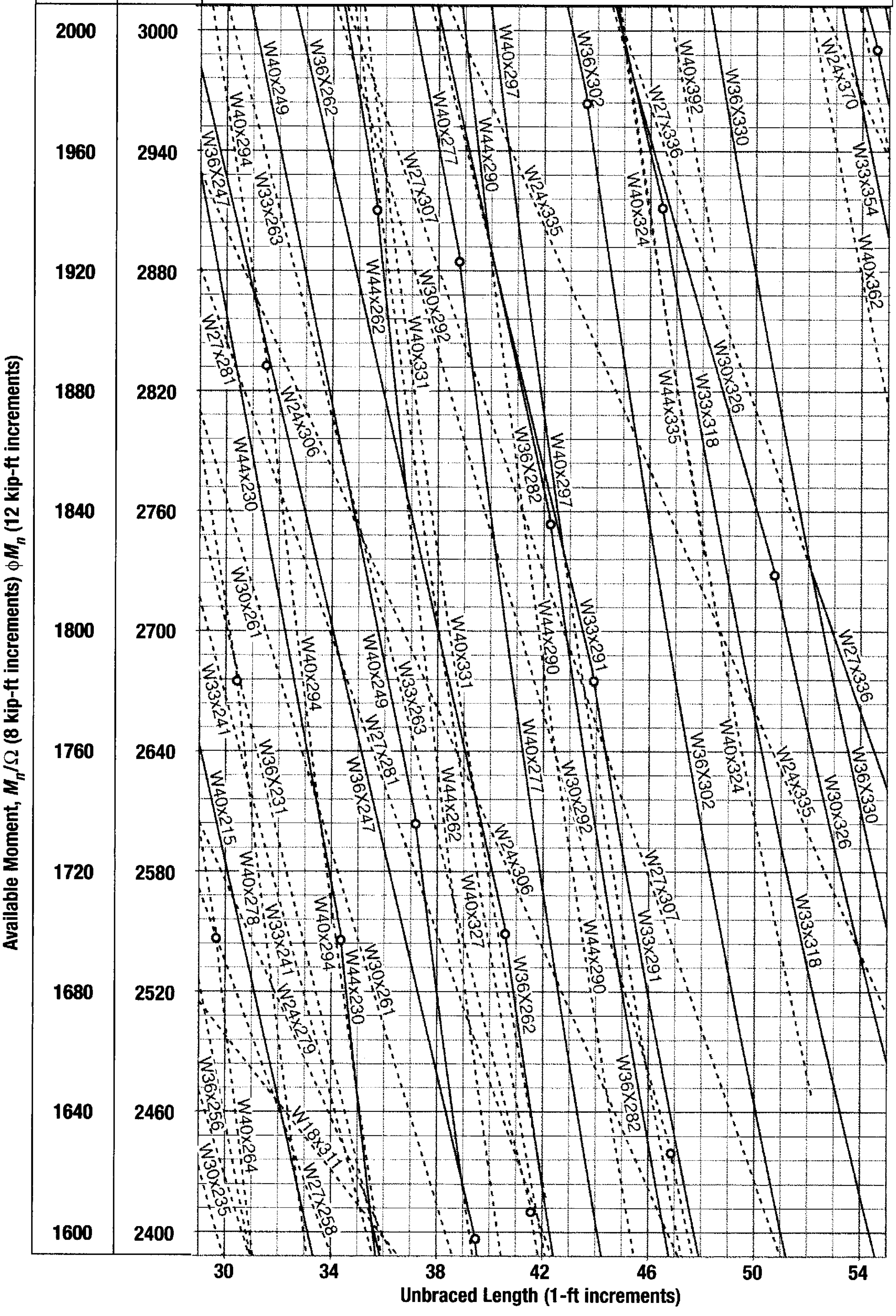


$F_y = 50$ ksi $C_b = 1$	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

W Shapes

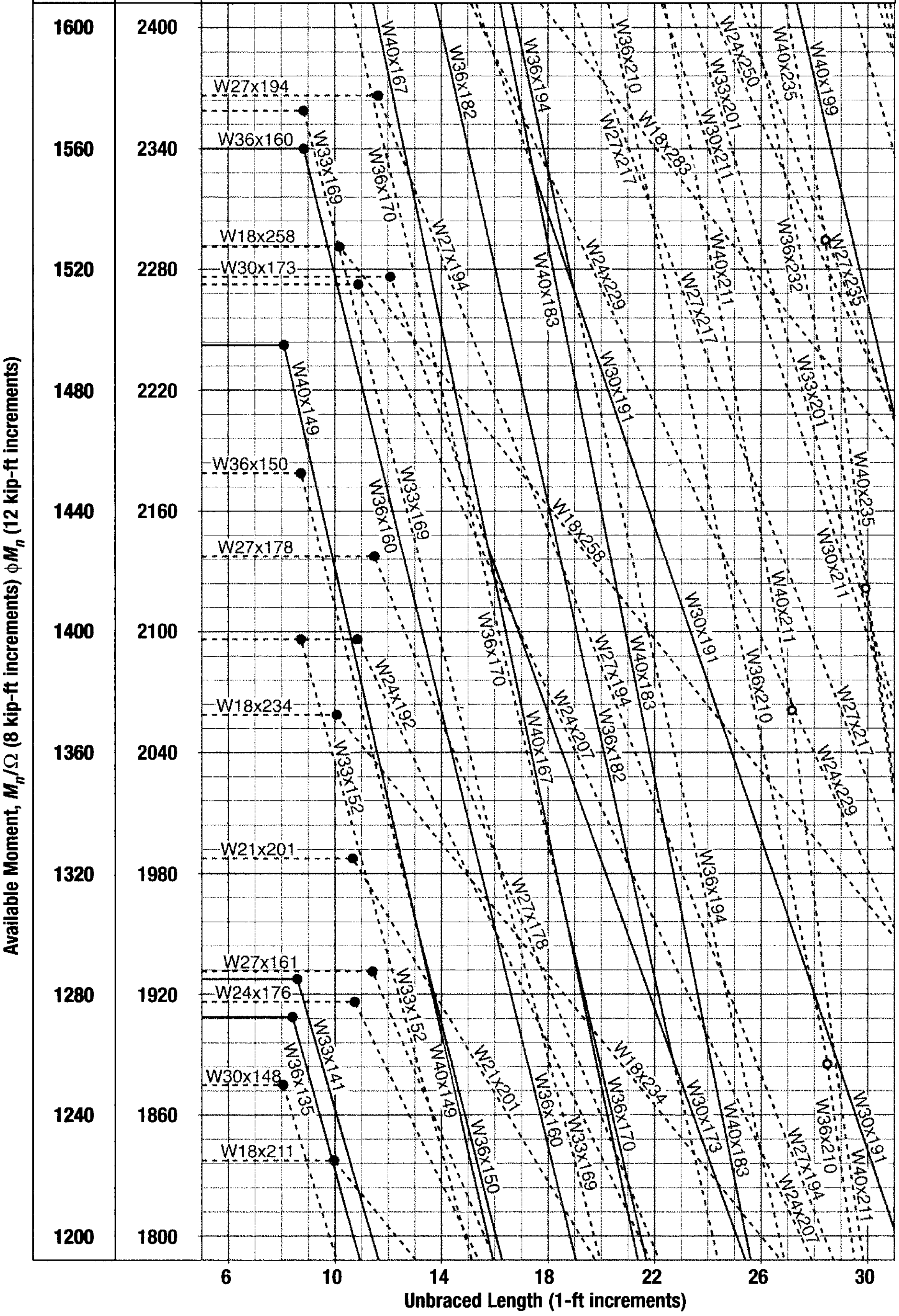
Available Moment vs. Unbraced Length



F_y = 50 ksi
C_b = 1

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

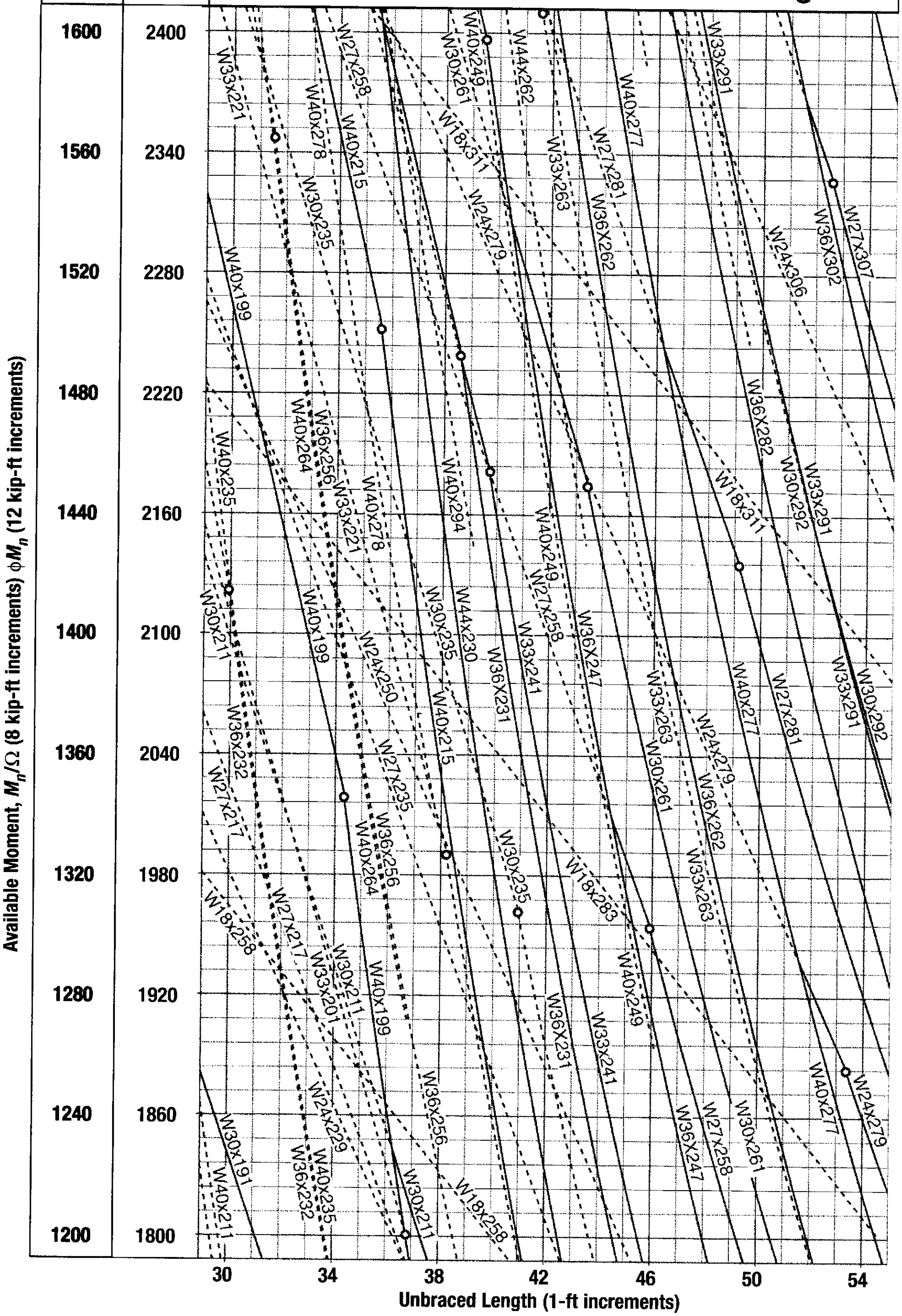
Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi
C_b = 1

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

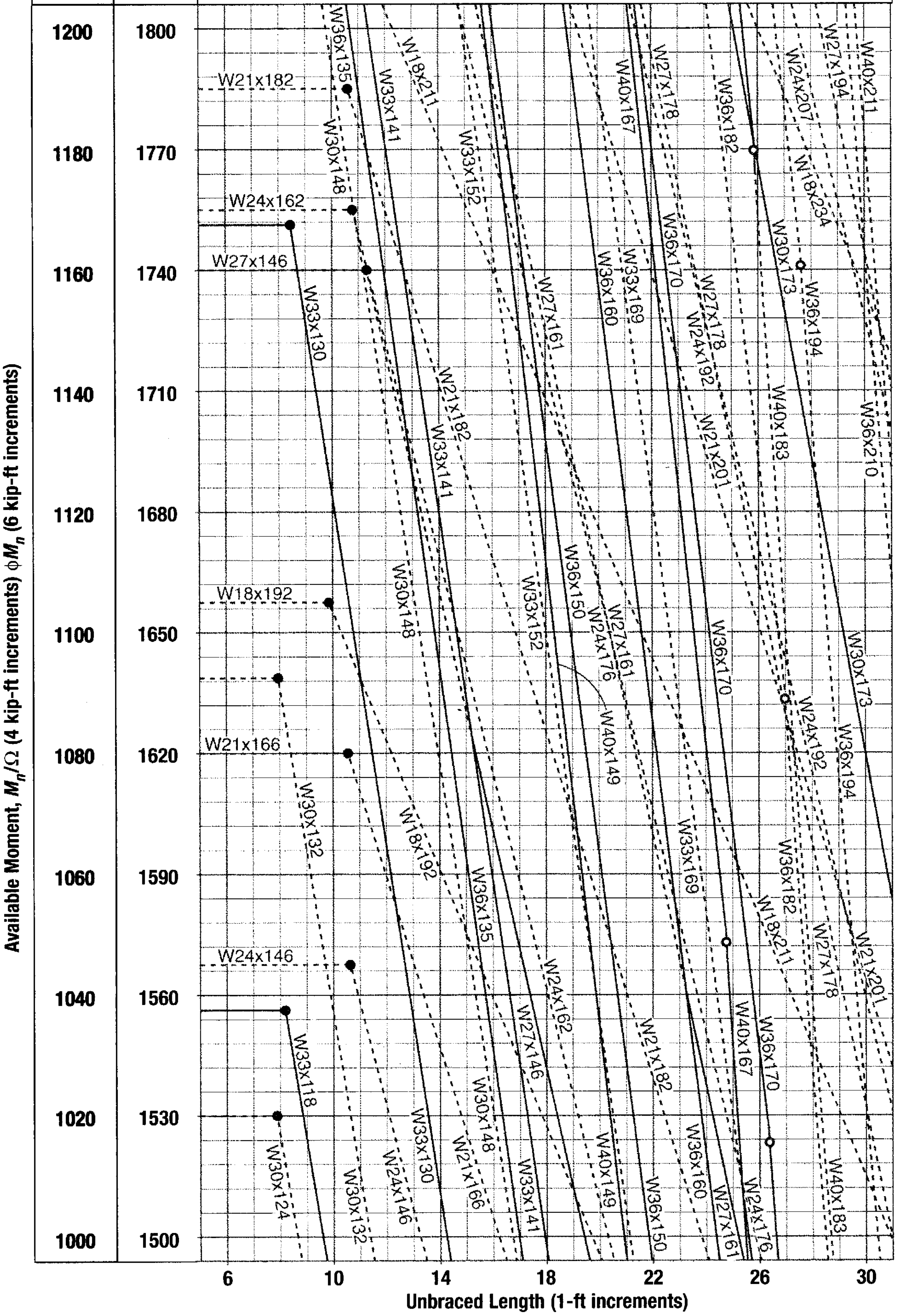
Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



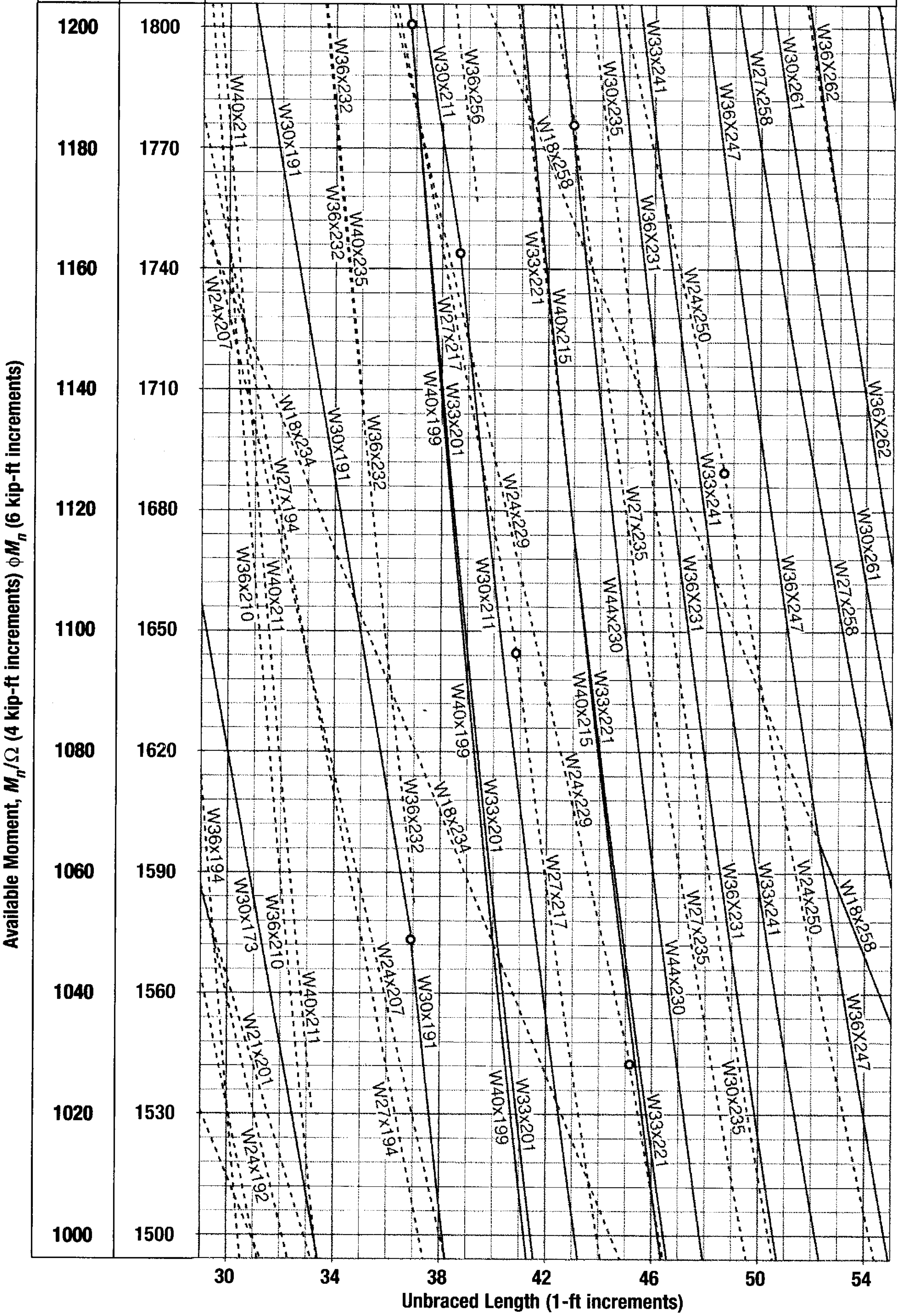
F_y = 50 ksi
C_b = 1

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

W Shapes

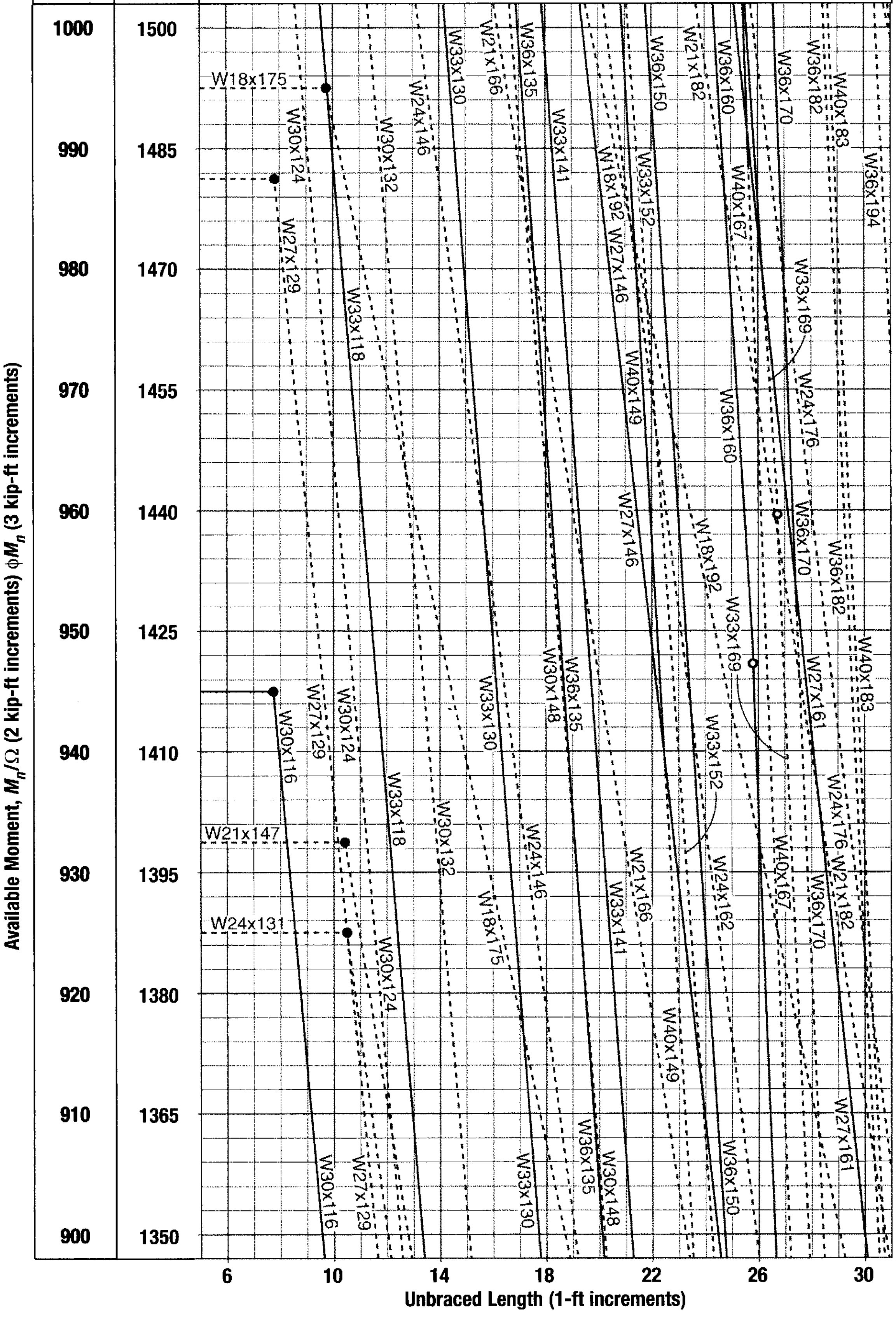
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

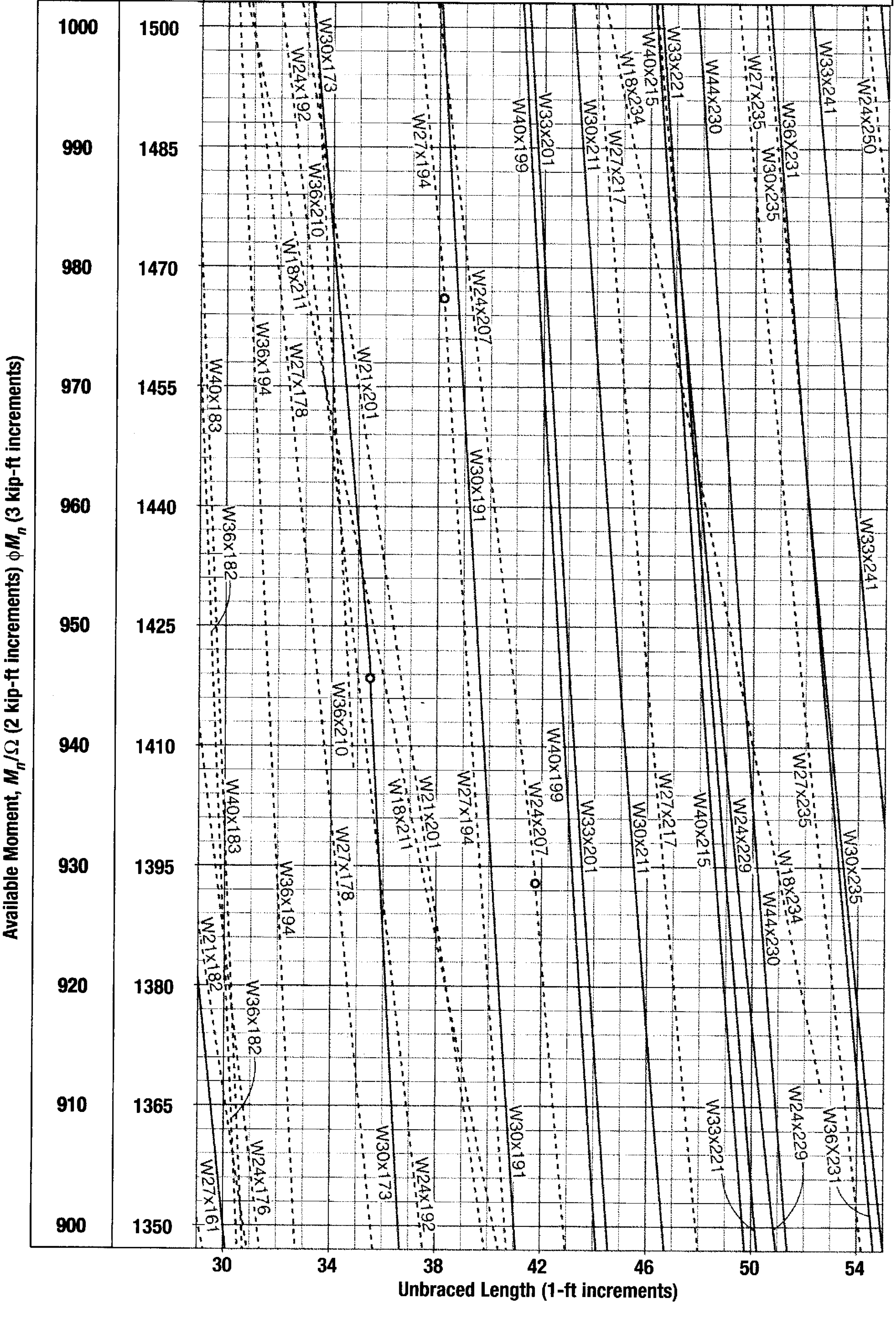
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi
C_b = 1

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length

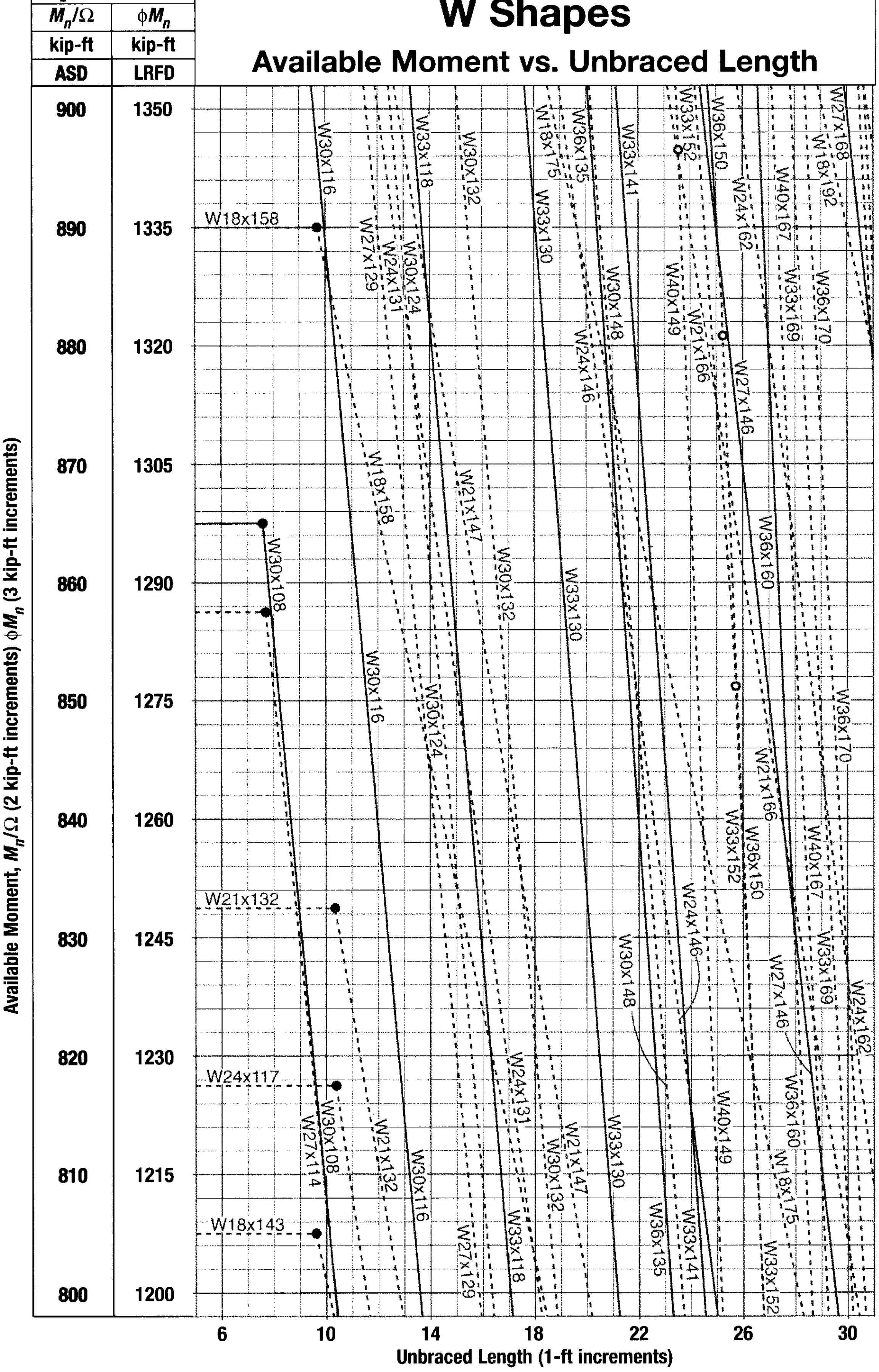
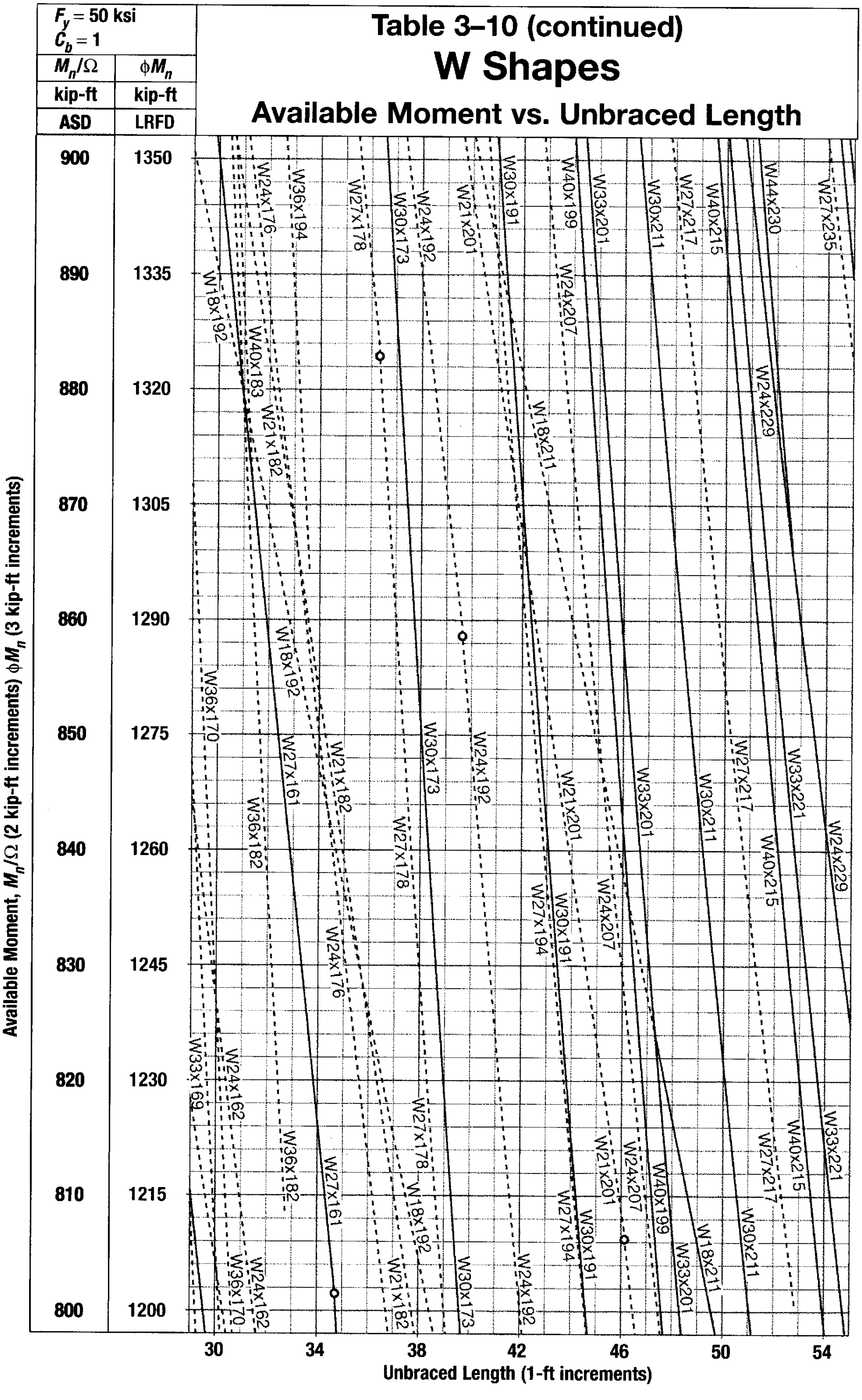


Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi
C_b = 1

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

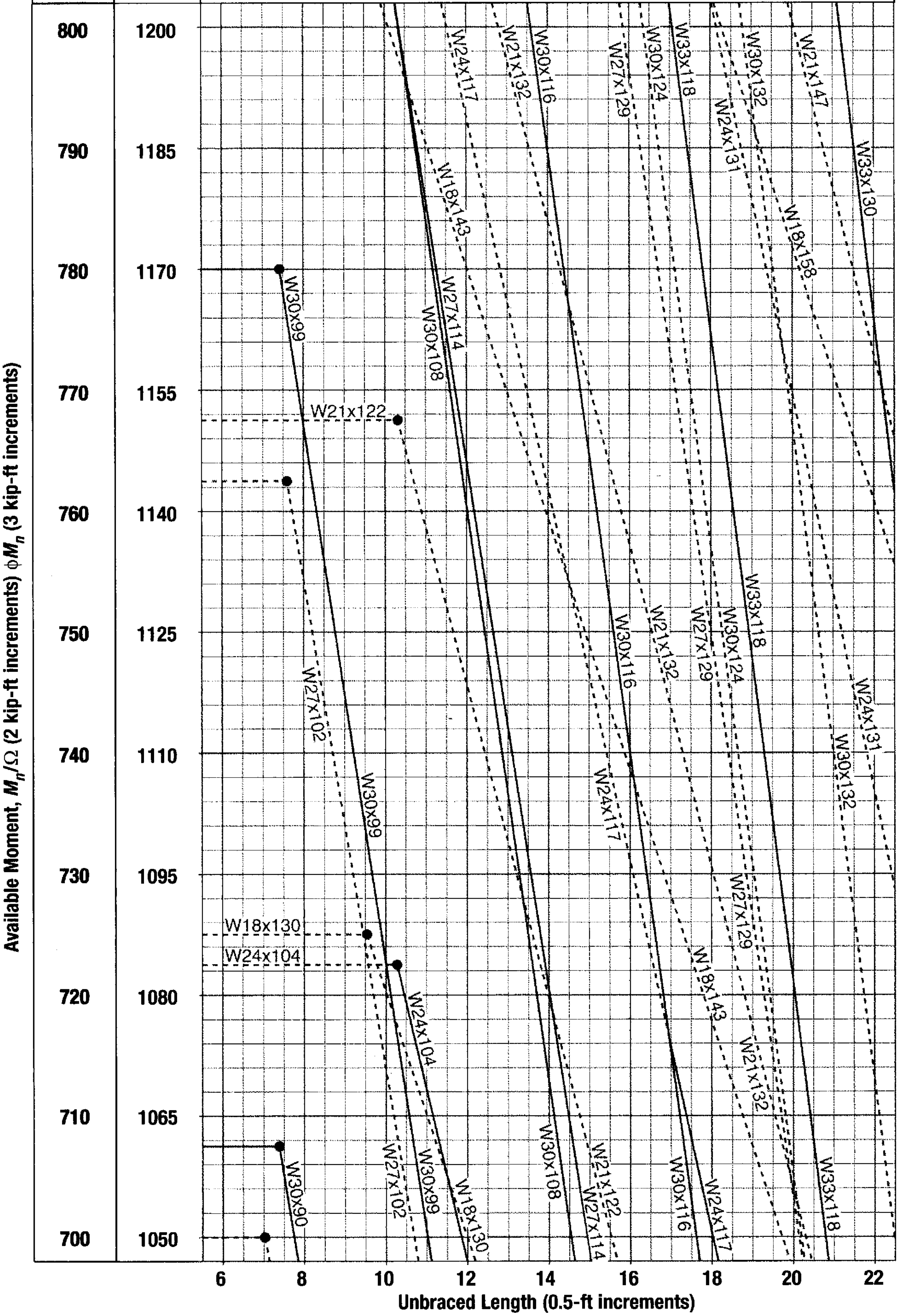
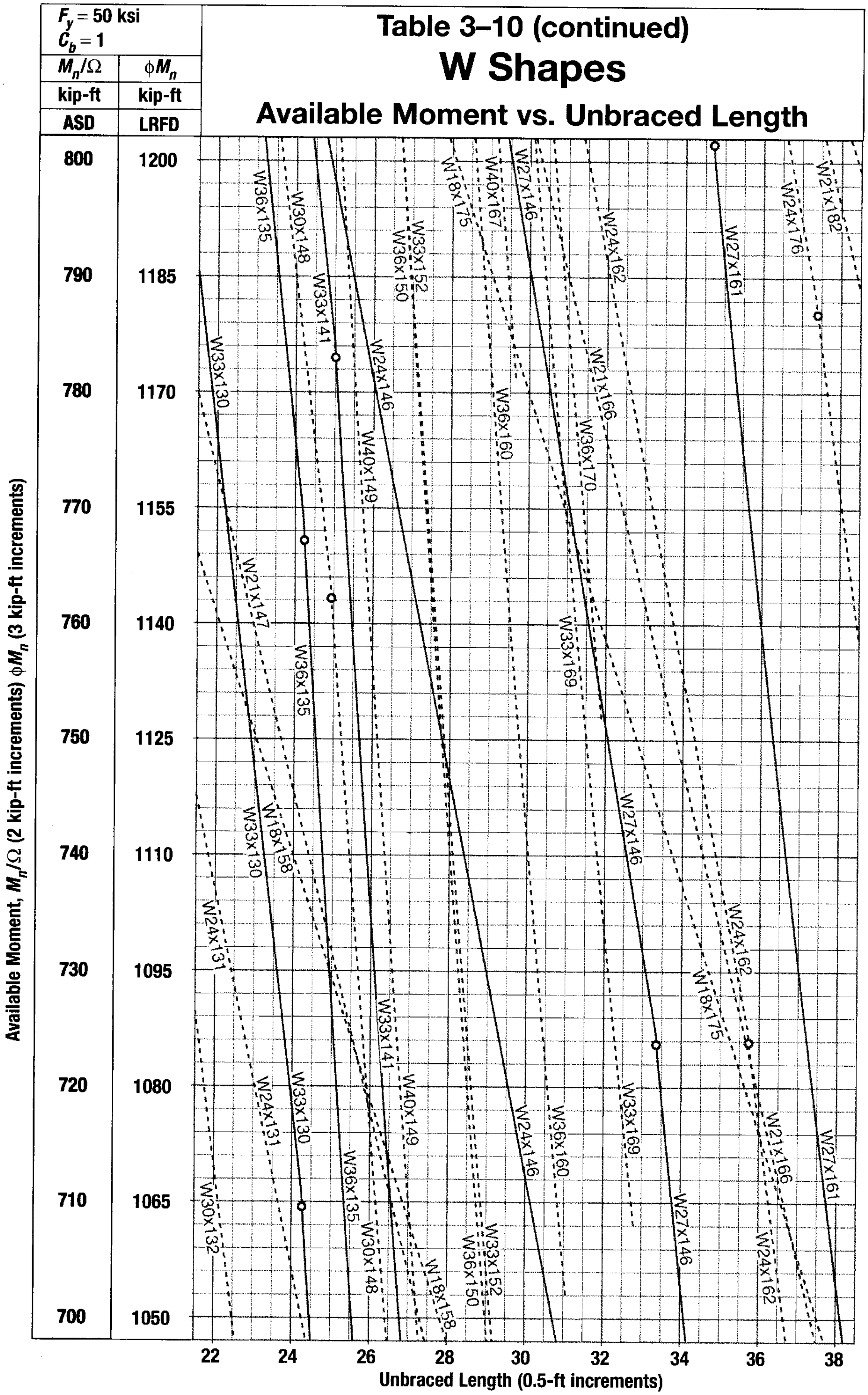


Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

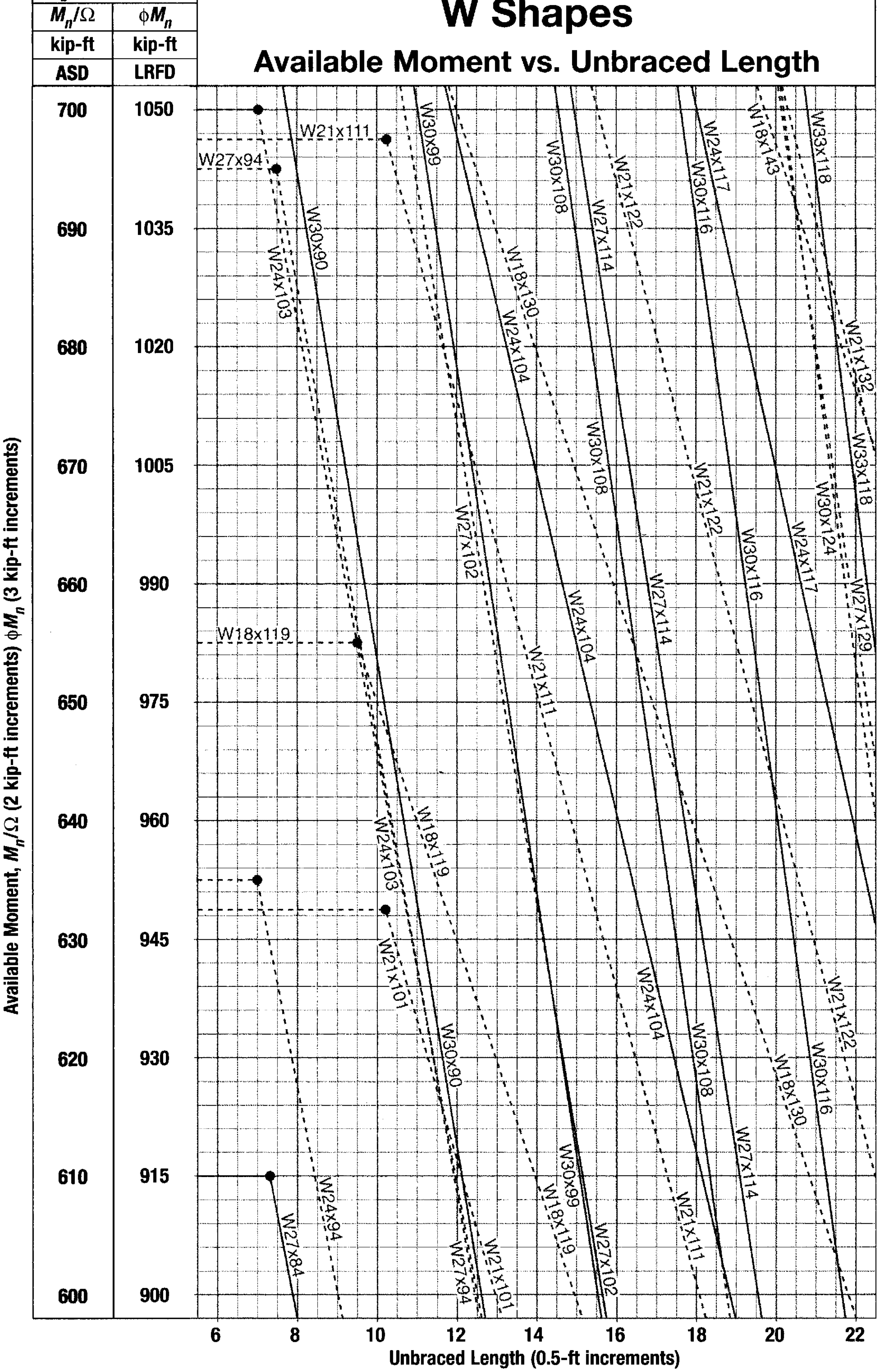
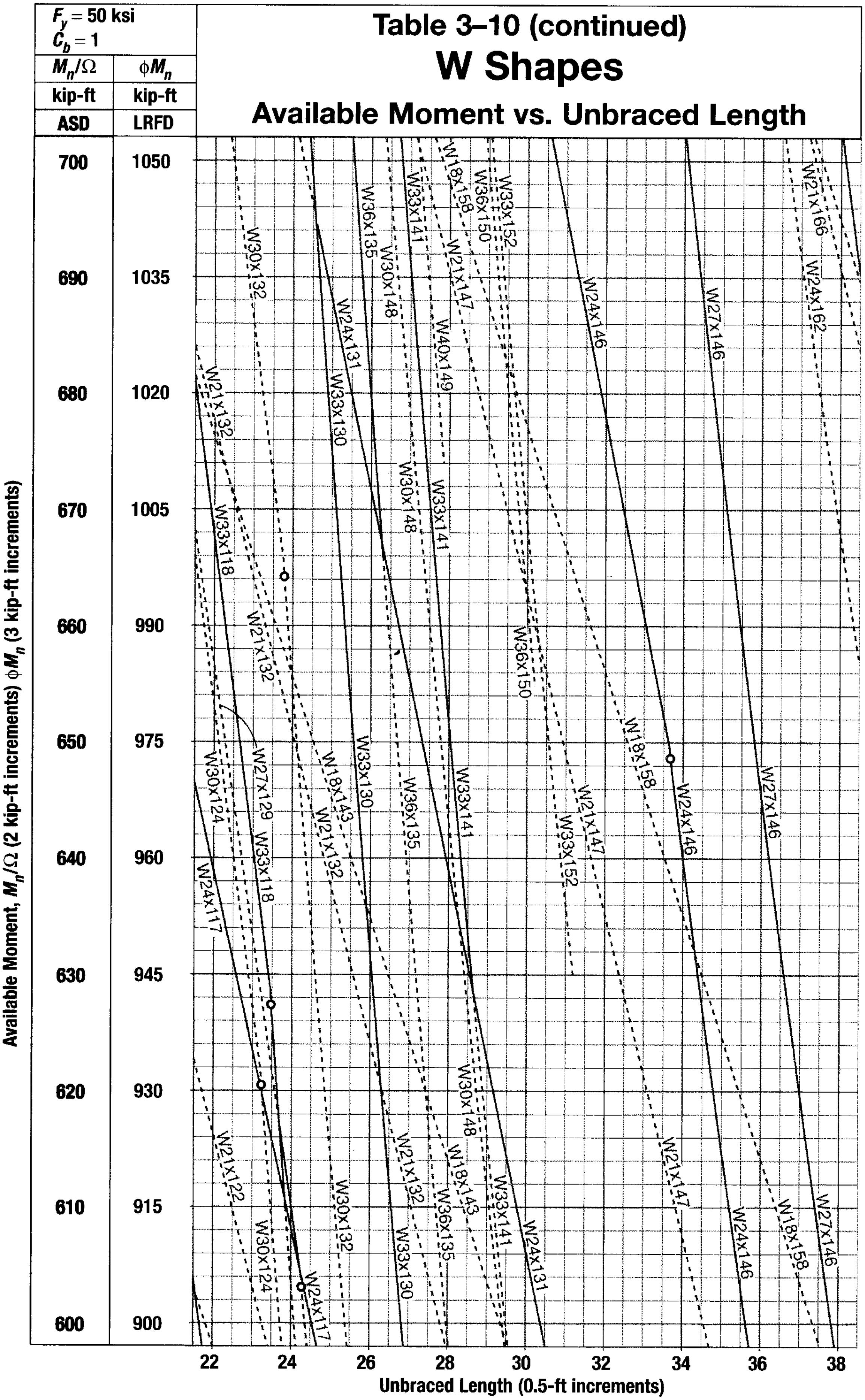


Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length

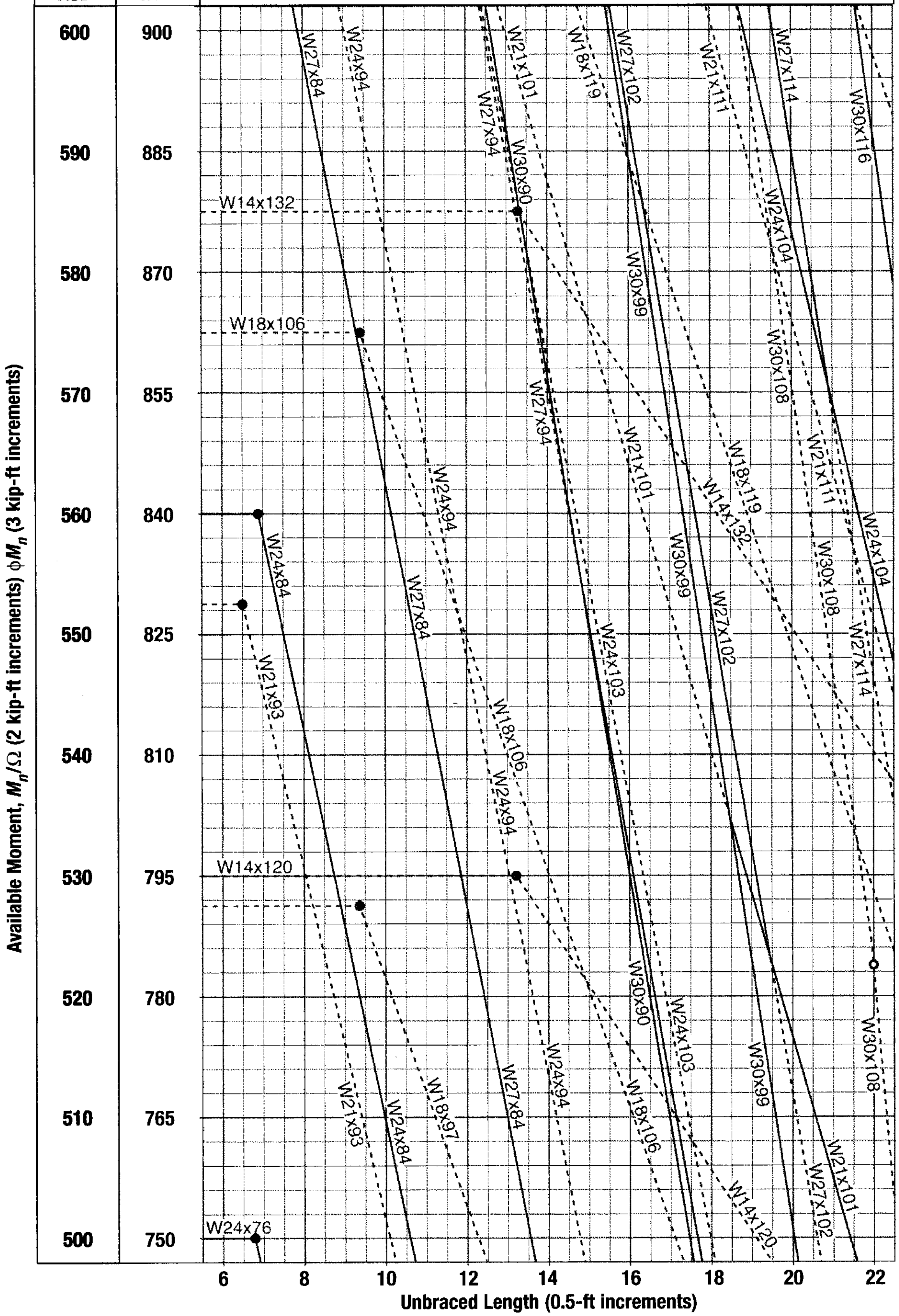


$F_y = 50$ ksi $C_b = 1$	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

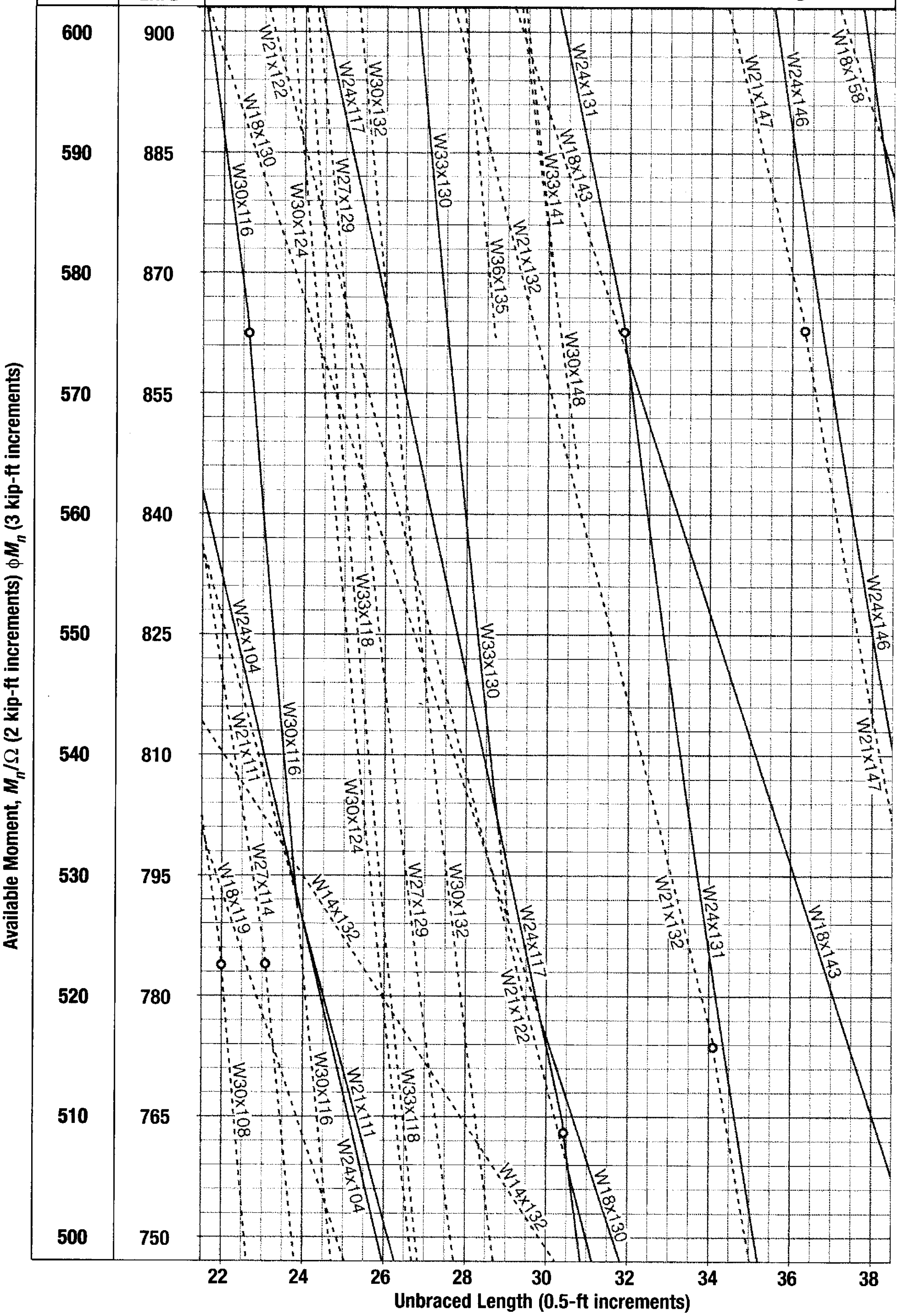
W Shapes

Available Moment vs. Unbraced Length



$F_y = 50$ ksi $C_b = 1$	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length

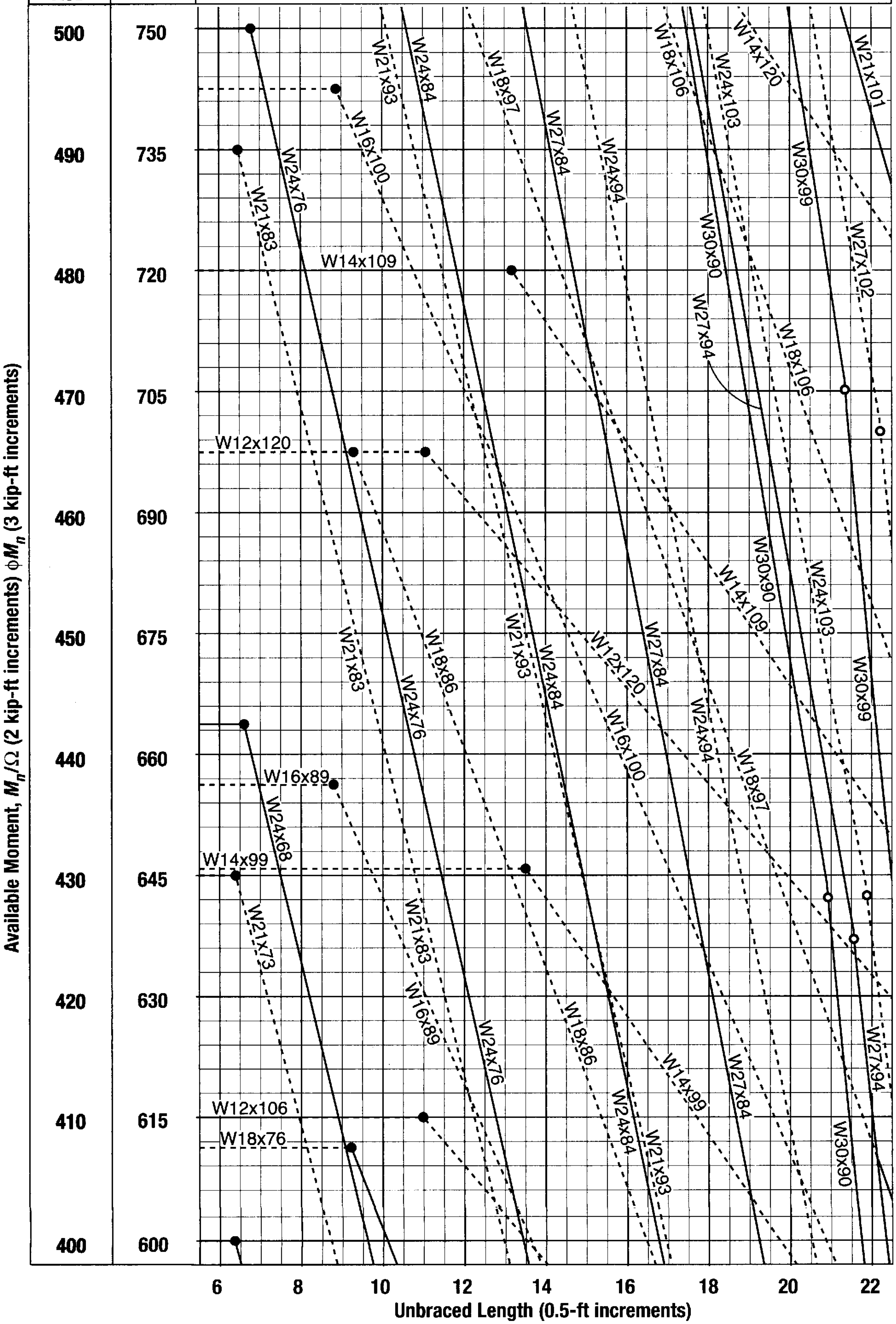
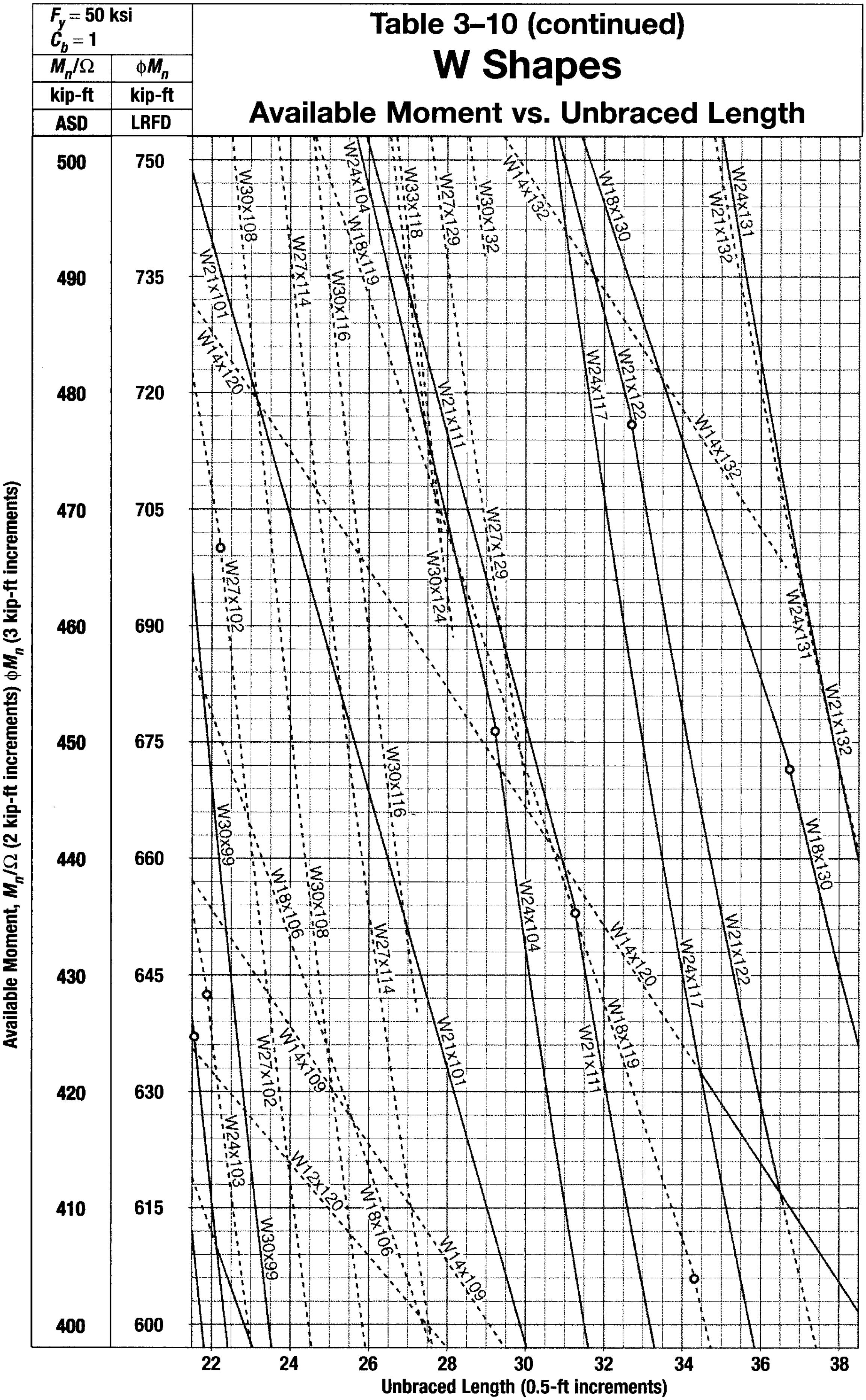


Table 3–10 (continued)

W Shapes

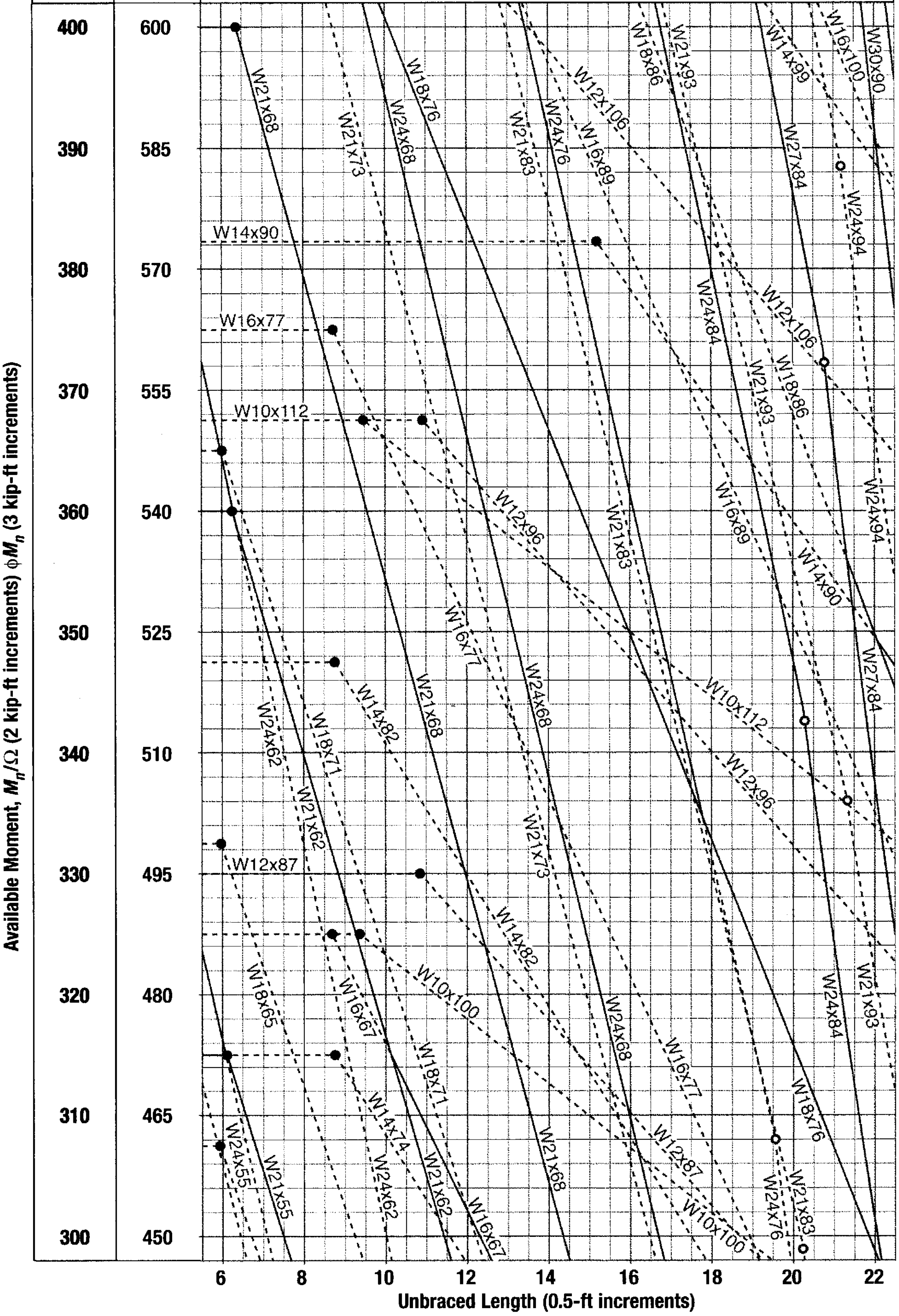
Available Moment vs. Unbraced Length



F_y = 50 ksi
C_b = 1

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

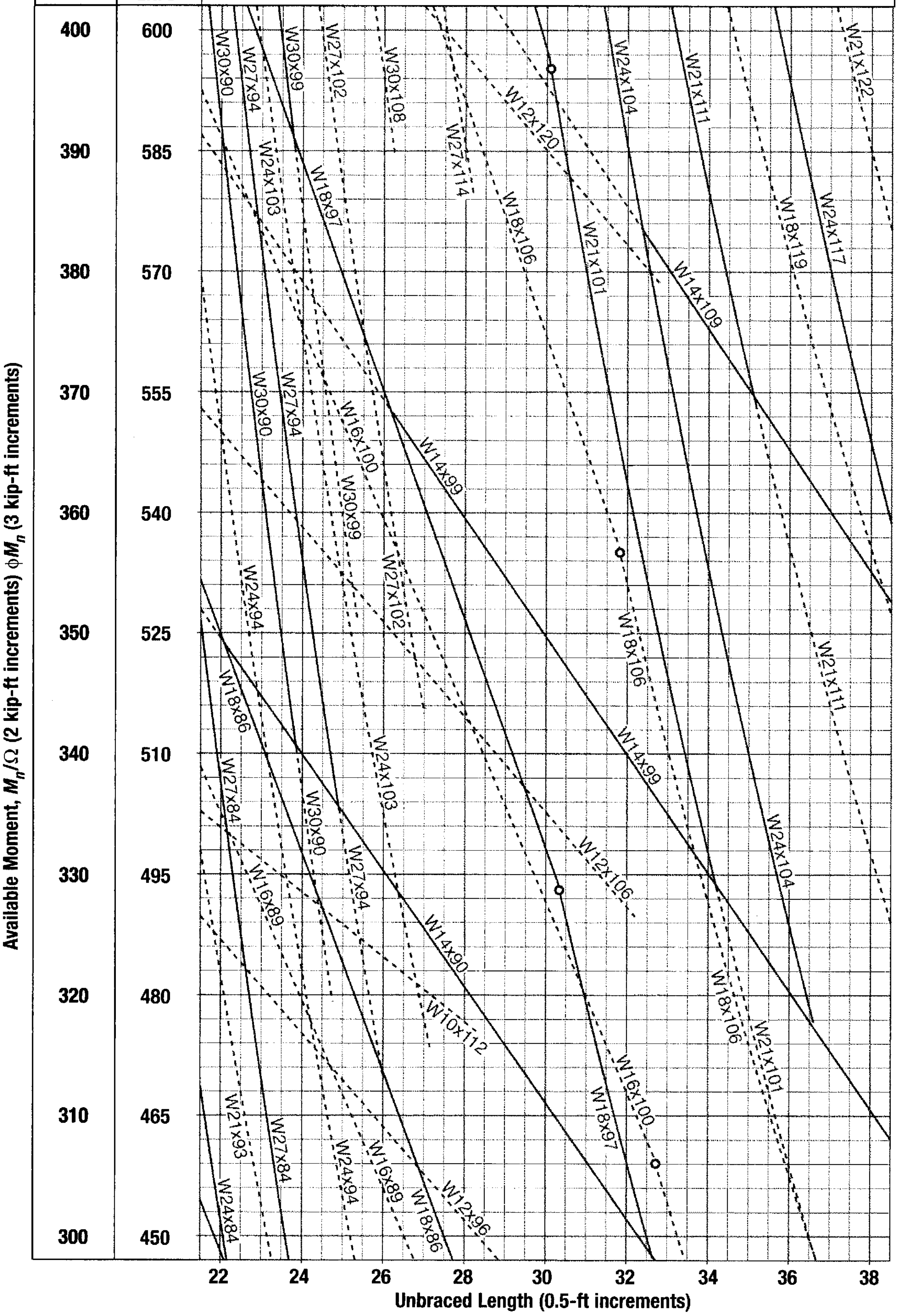


F_y = 50 ksi C_b = 1	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length



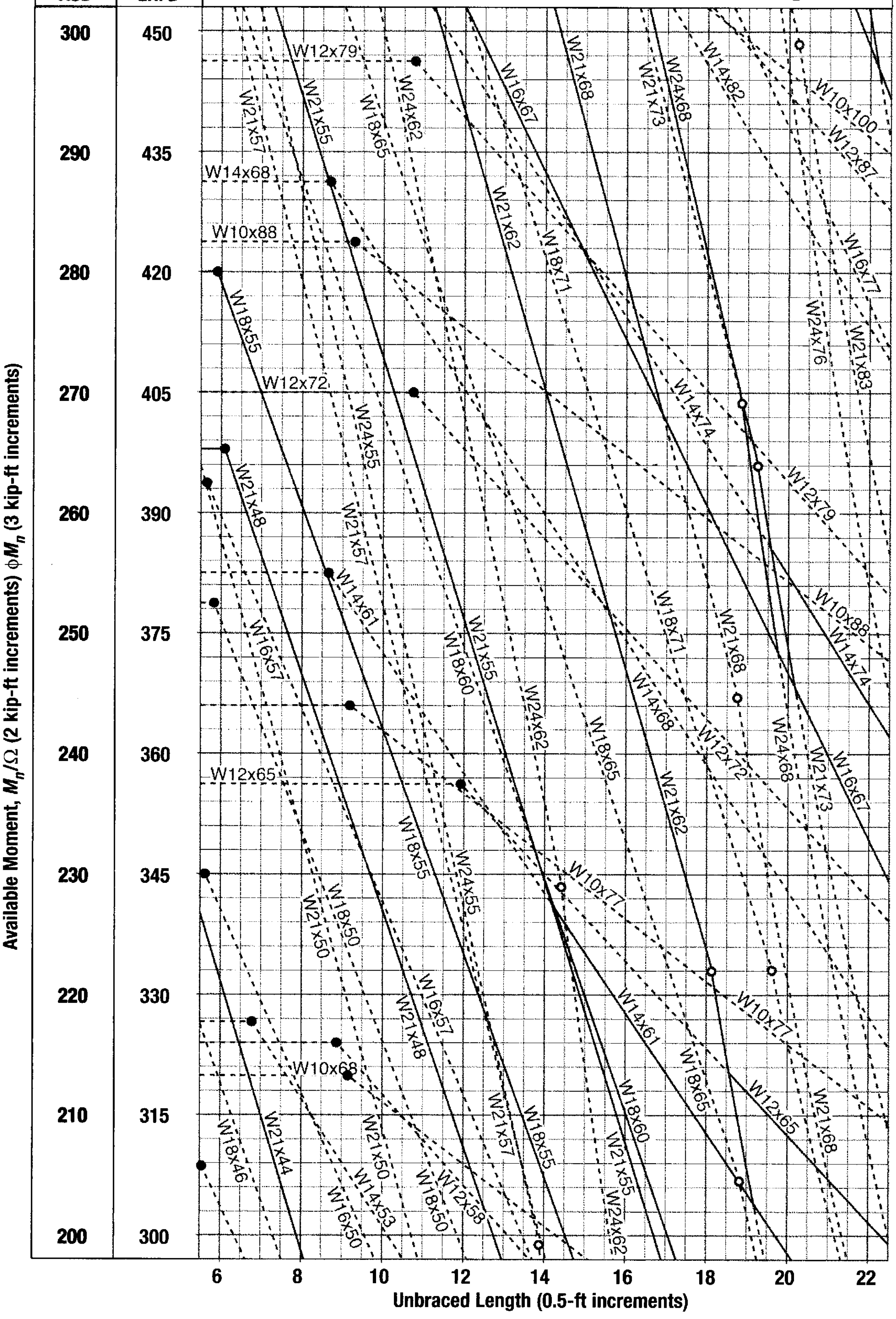
F_y = 50 ksi
C_b = 1

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

W Shapes

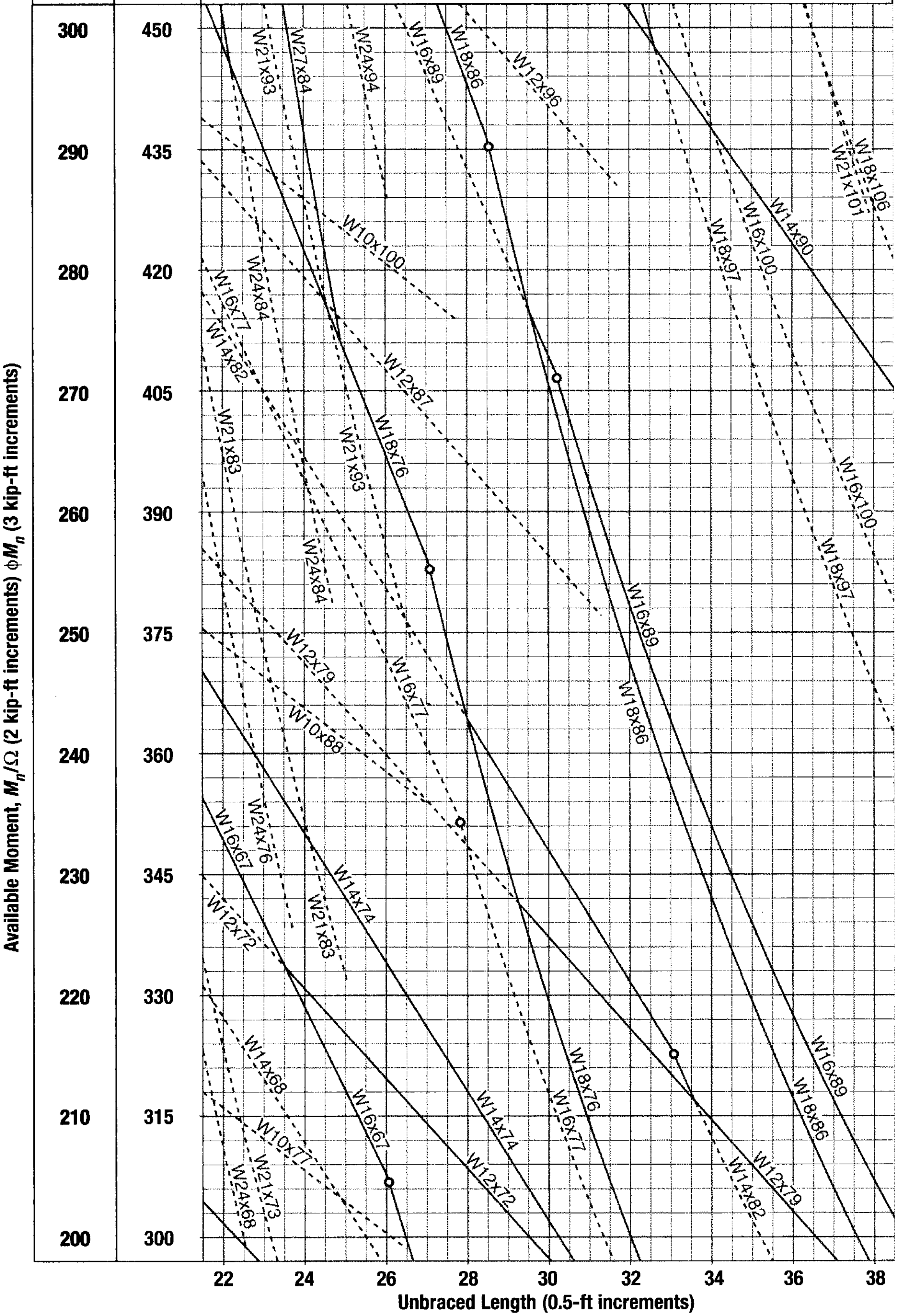
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

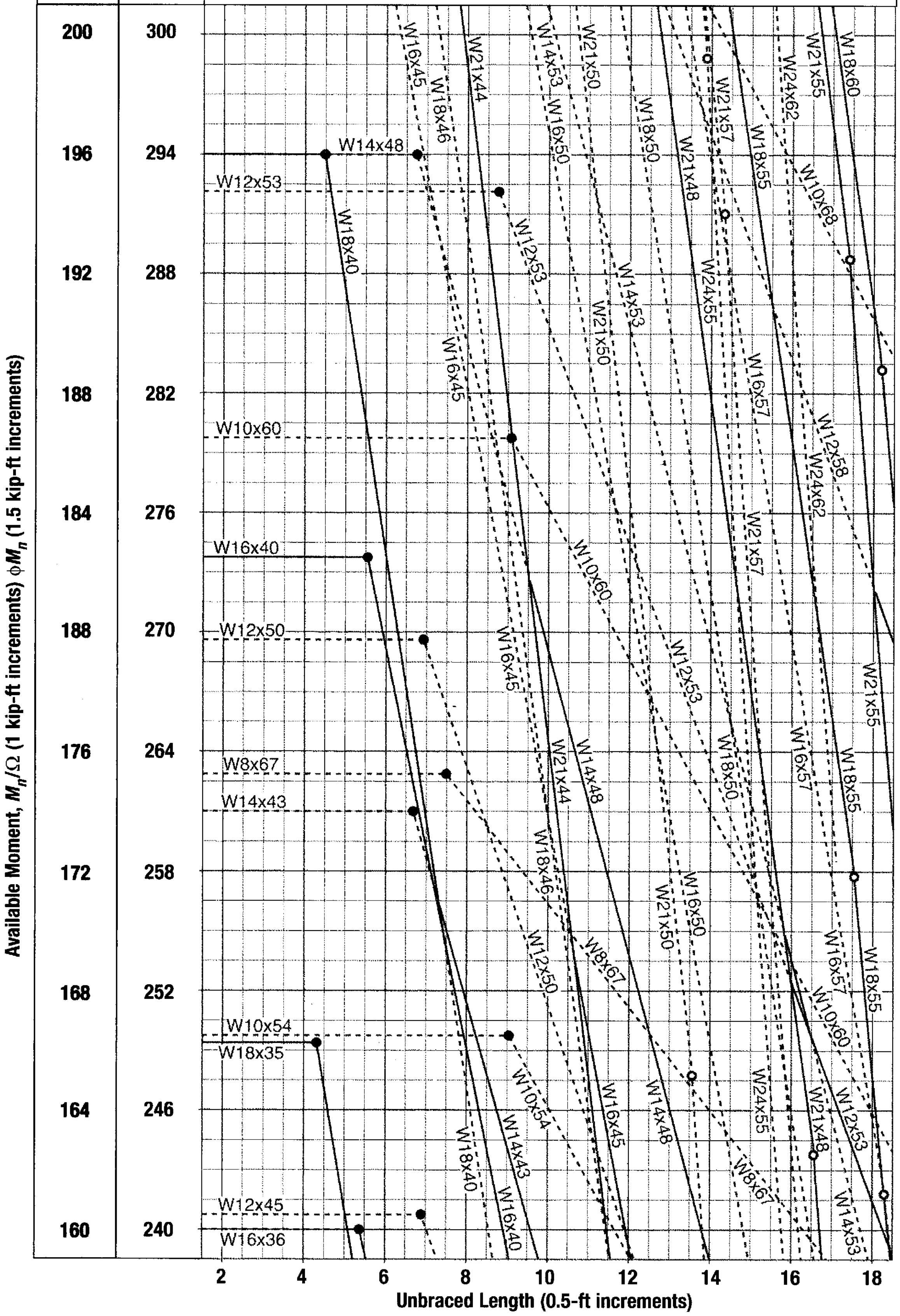


$F_y = 50$ ksi $C_b = 1$	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

W Shapes

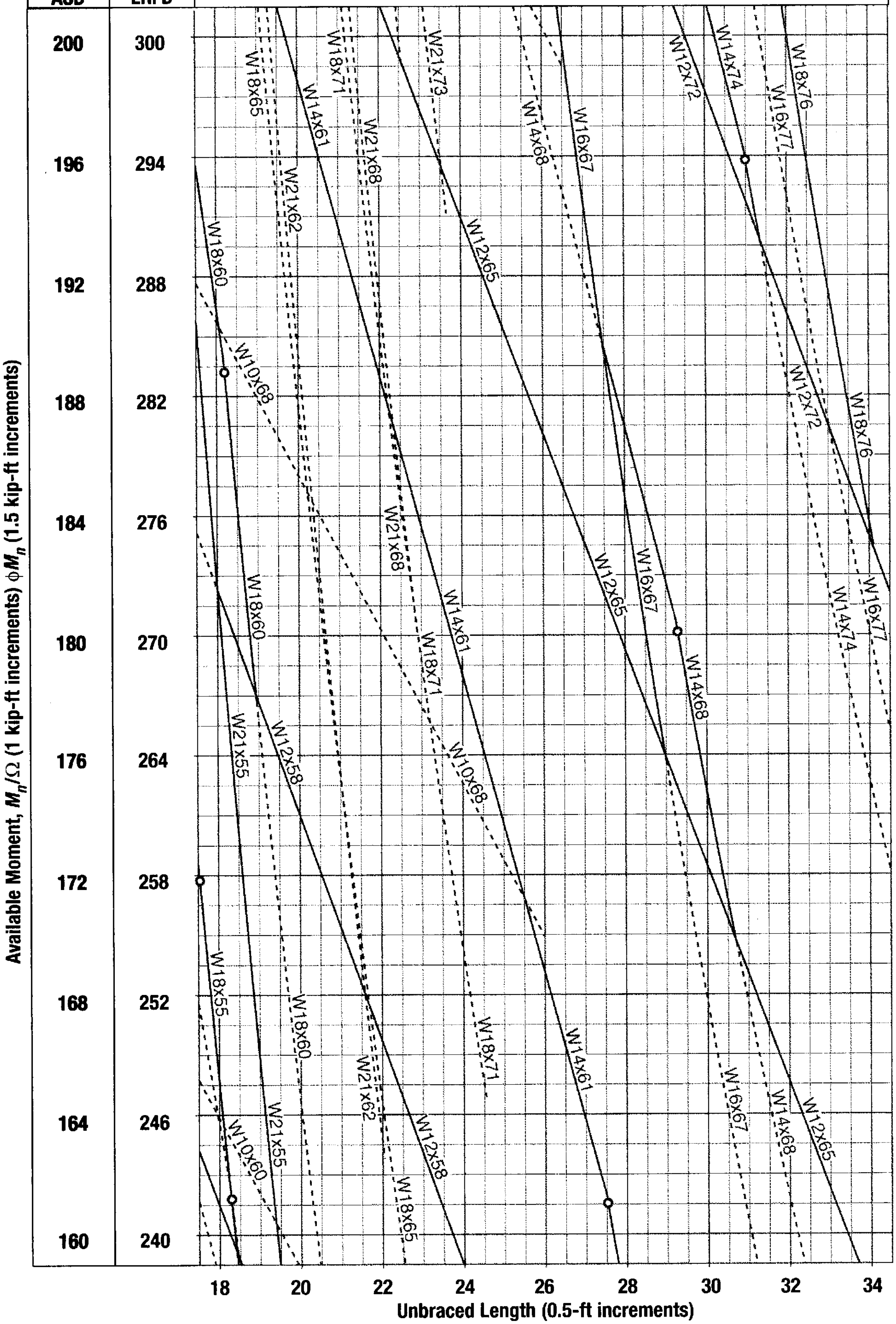
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

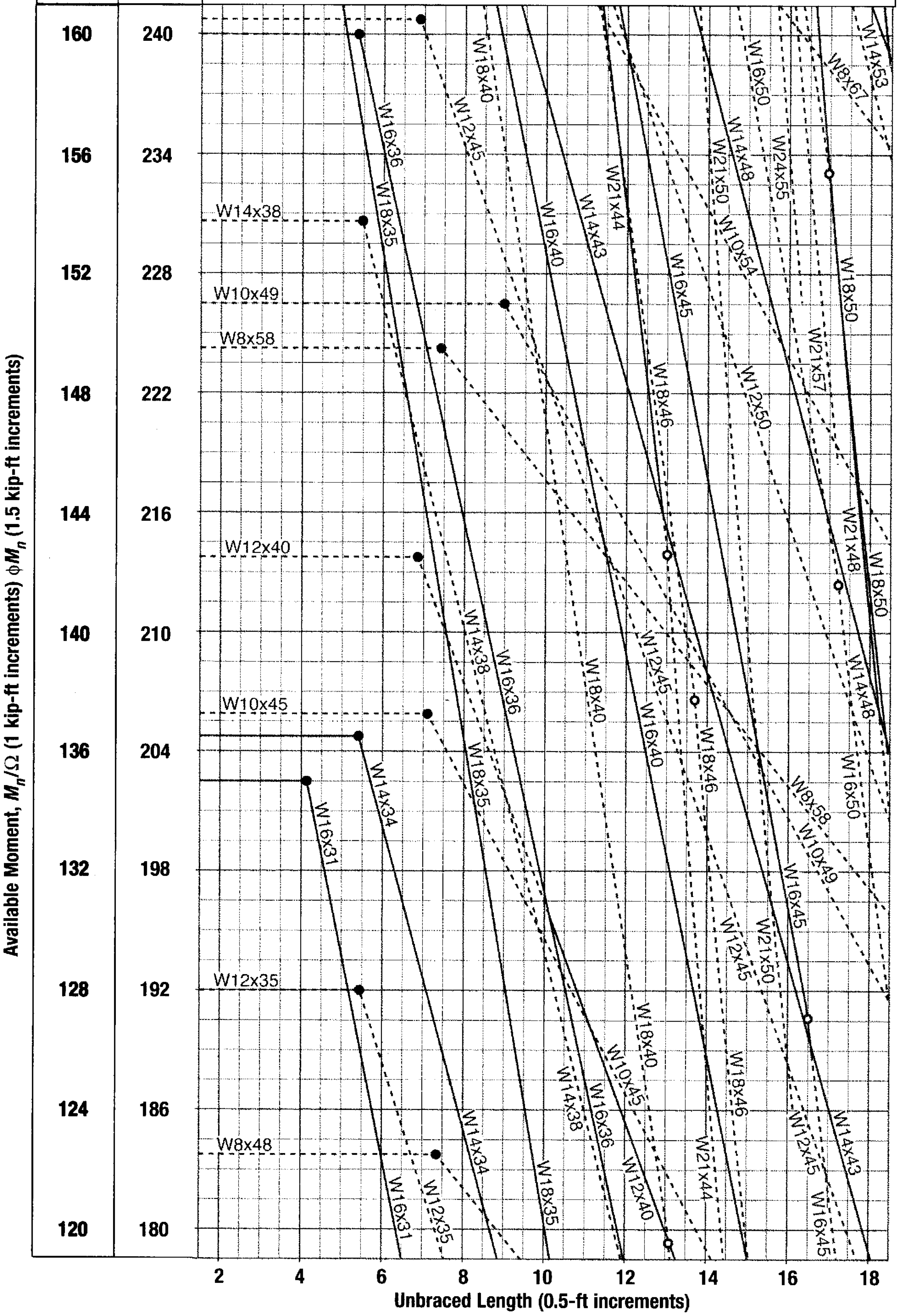


$F_y = 50$ ksi $C_b = 1$	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

W Shapes

Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

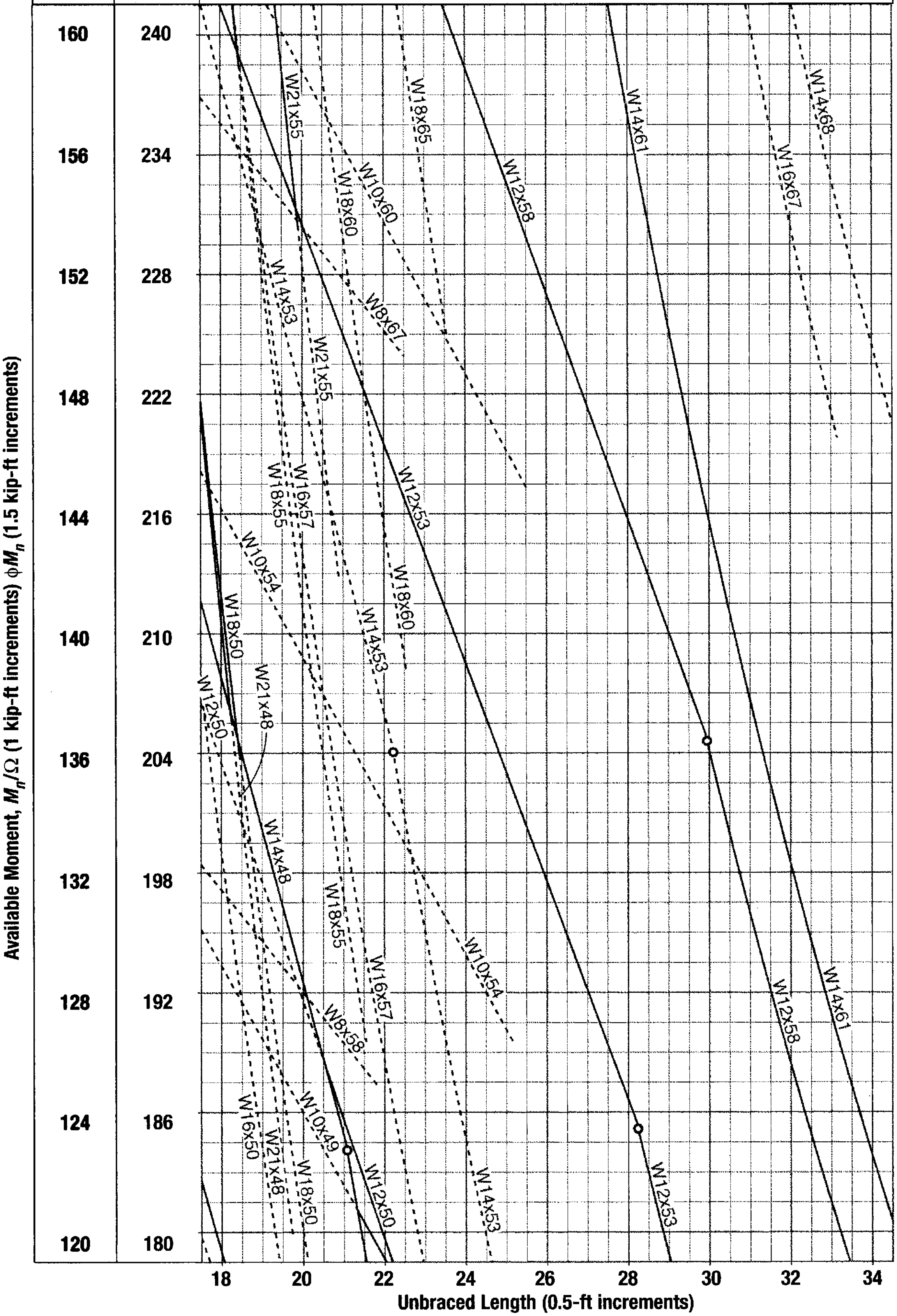
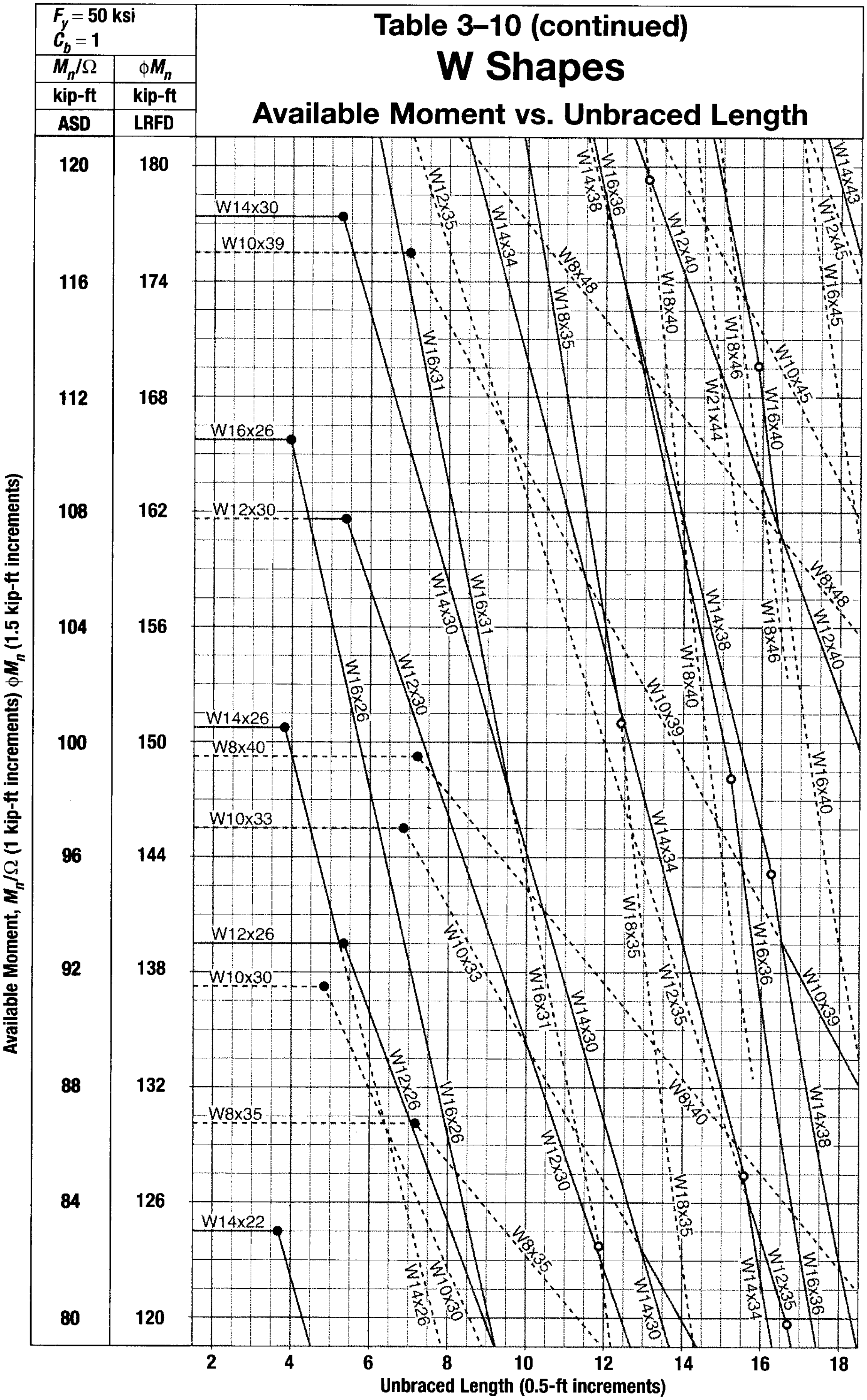


Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

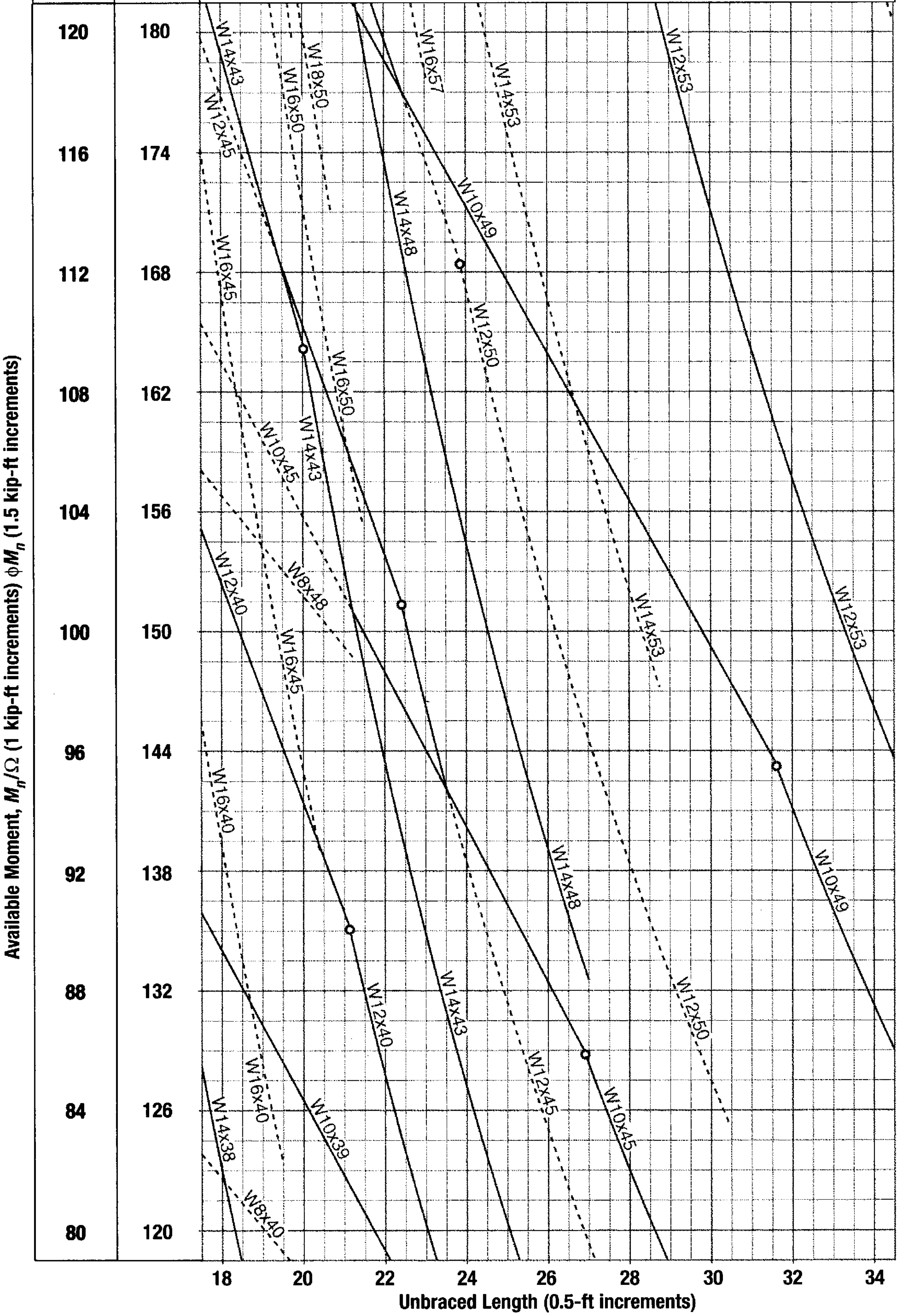


F_y = 50 ksi C_b = 1	
M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)

W Shapes

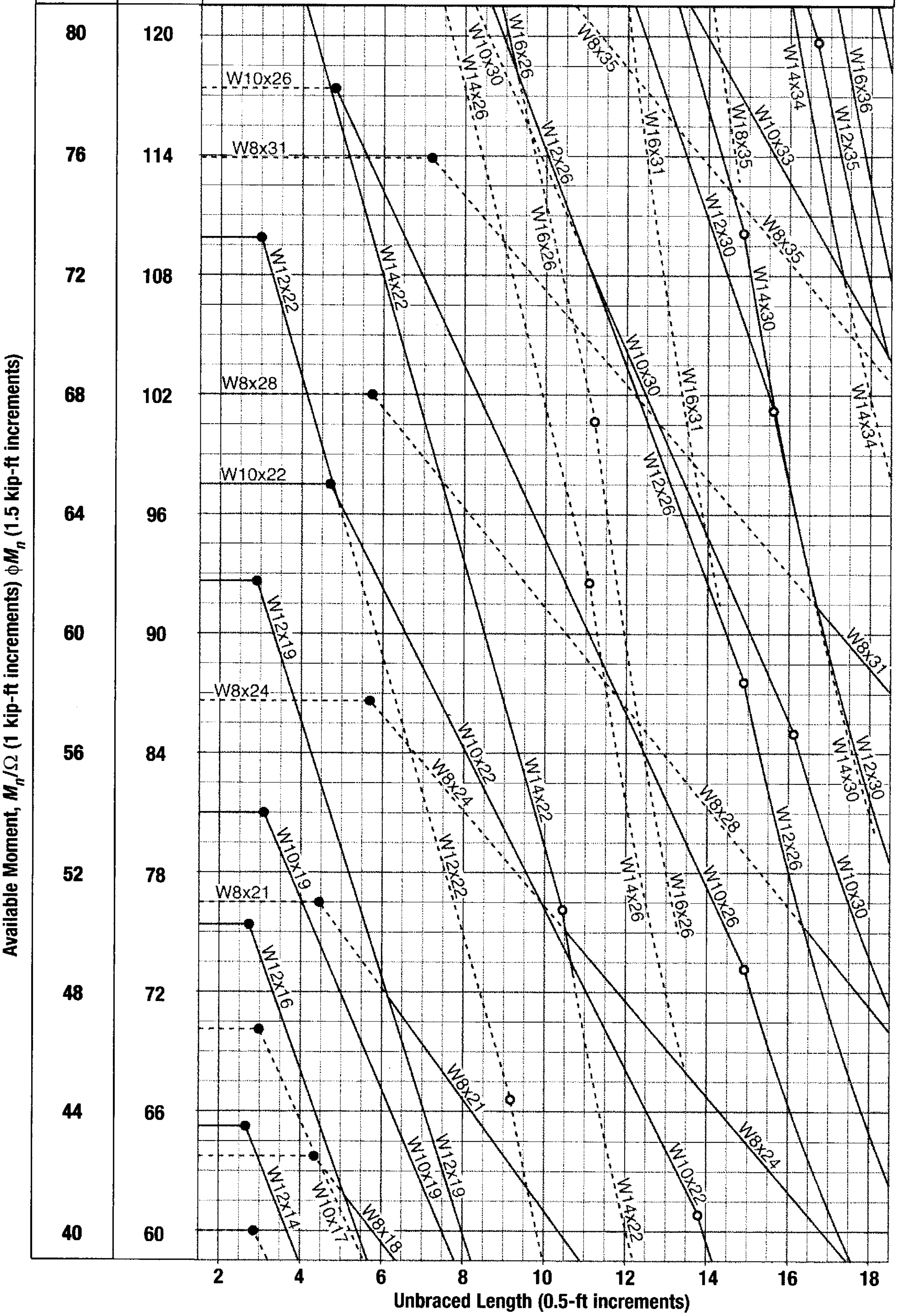
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

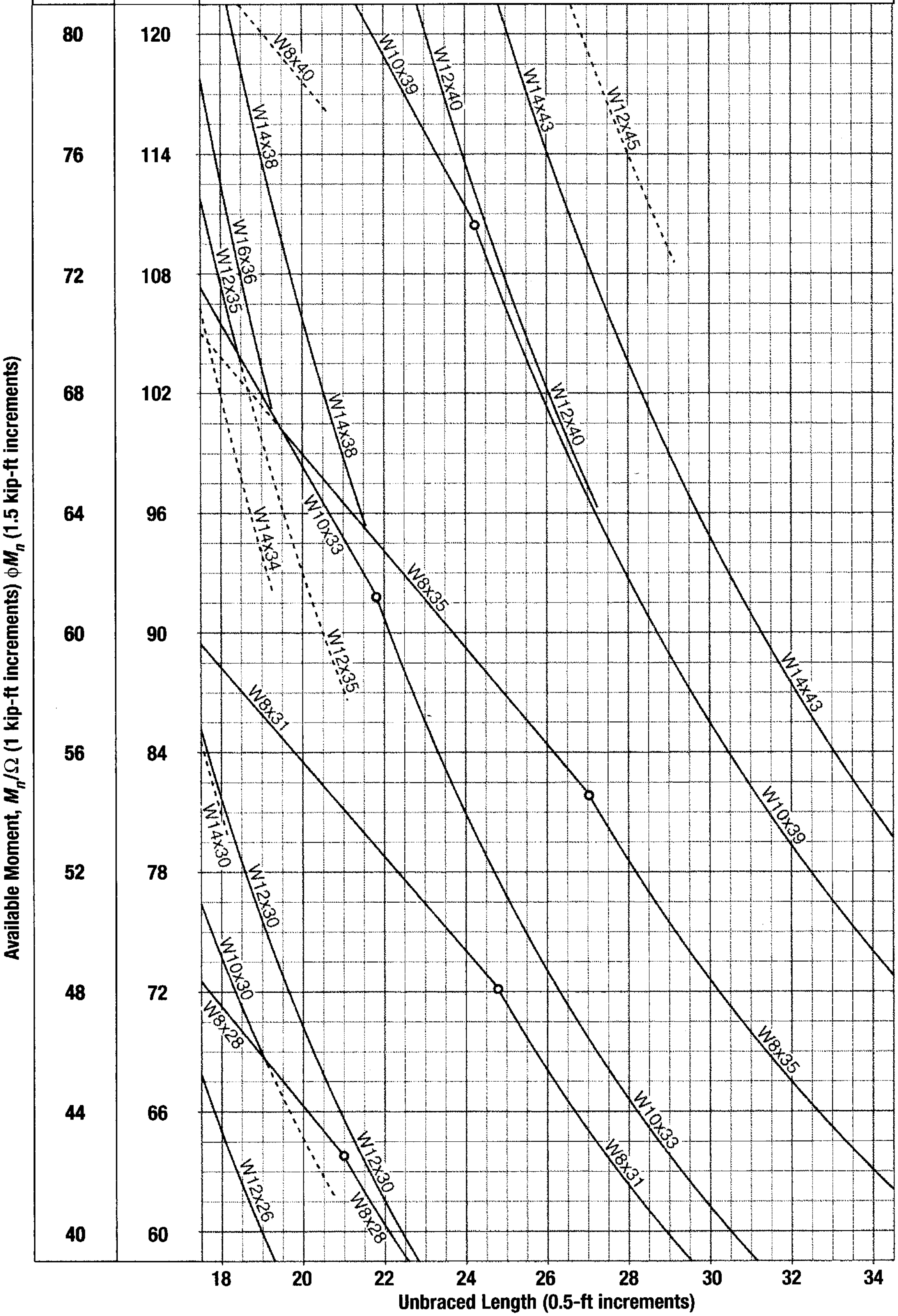
Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



$F_y = 50$ ksi
$C_b = 1$

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length



F_y = 50 ksi
C_b = 1

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–10 (continued)
W Shapes
Available Moment vs. Unbraced Length

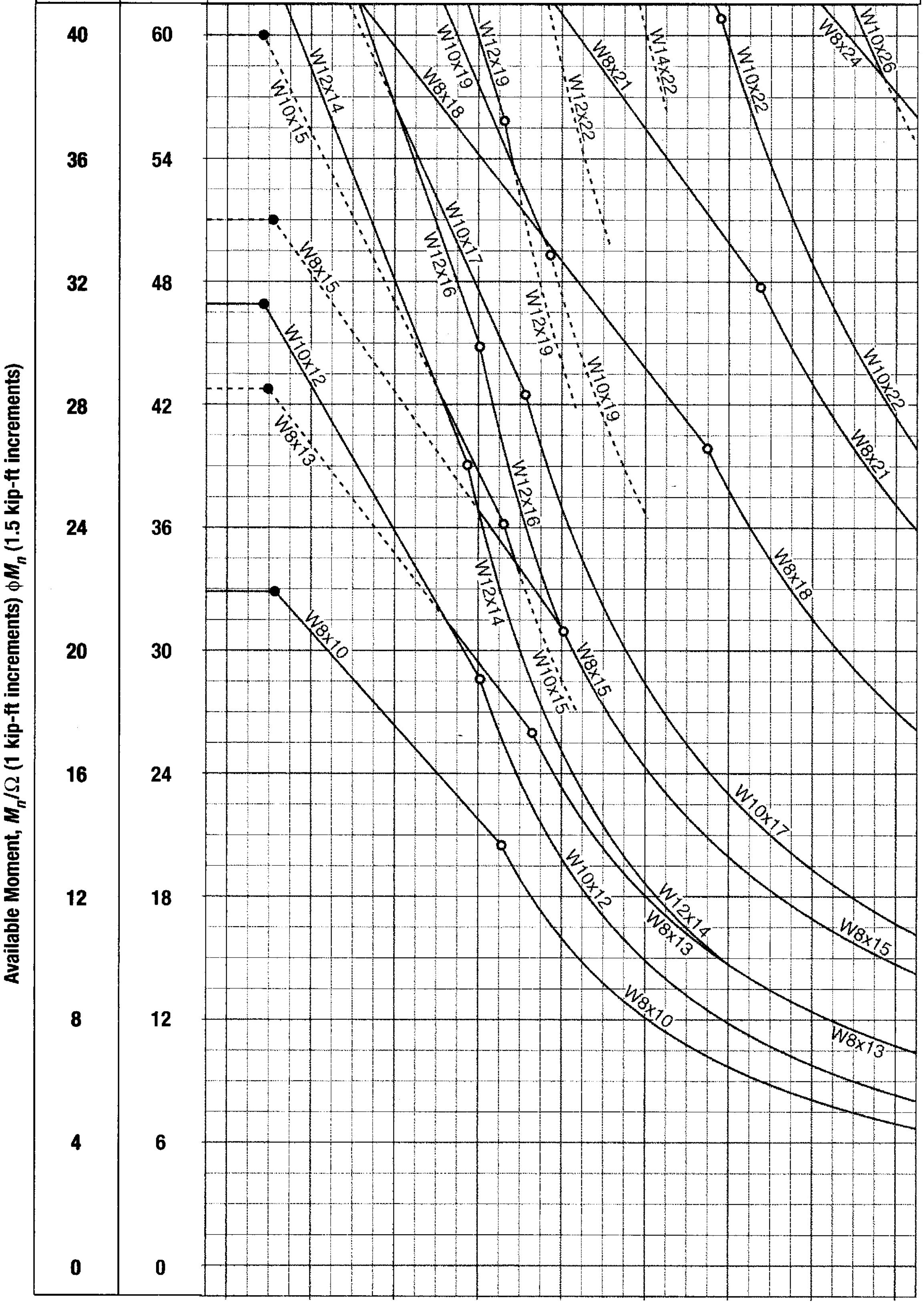


Table 3–11

Channels

Available Moment vs. Unbraced Length

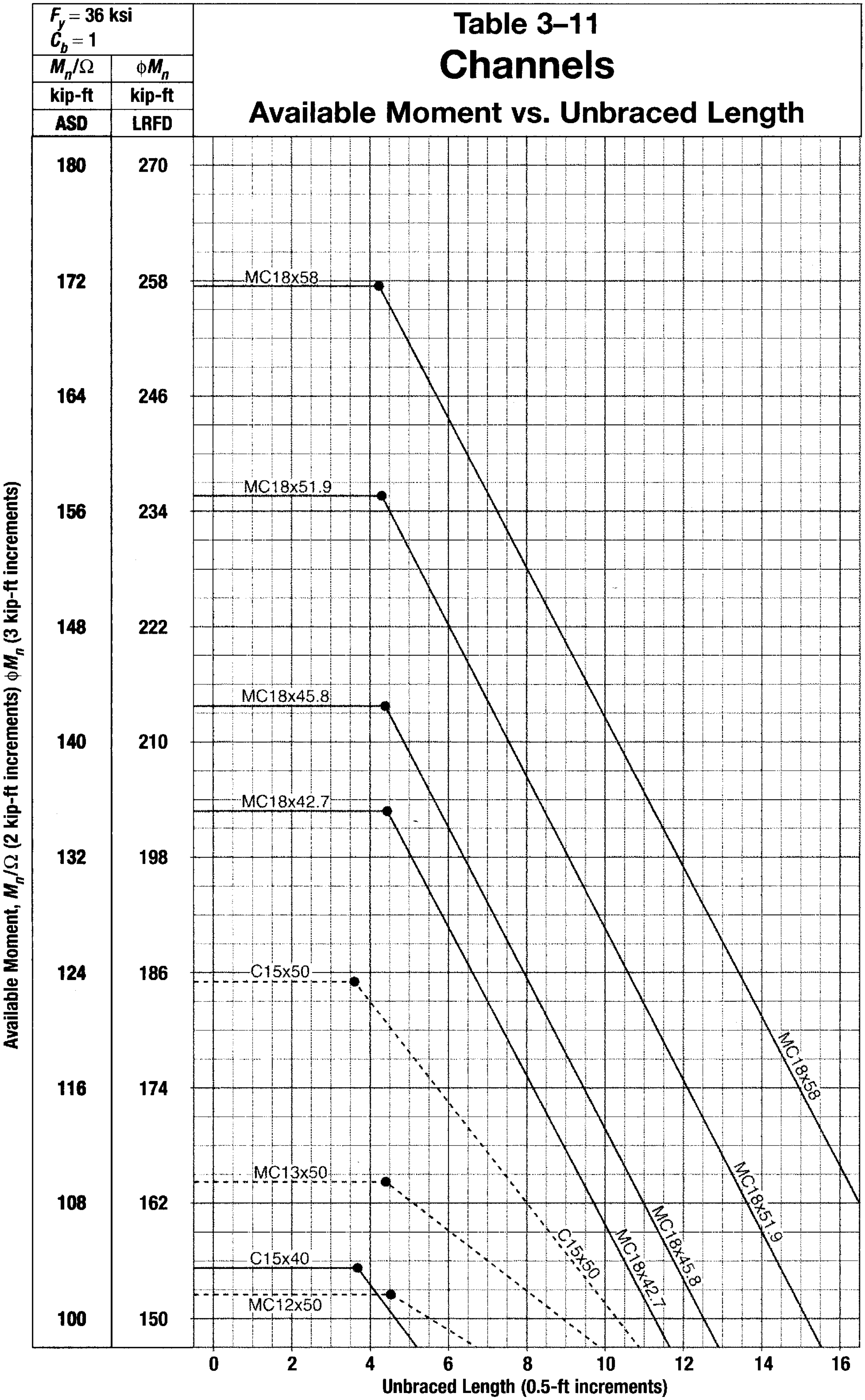
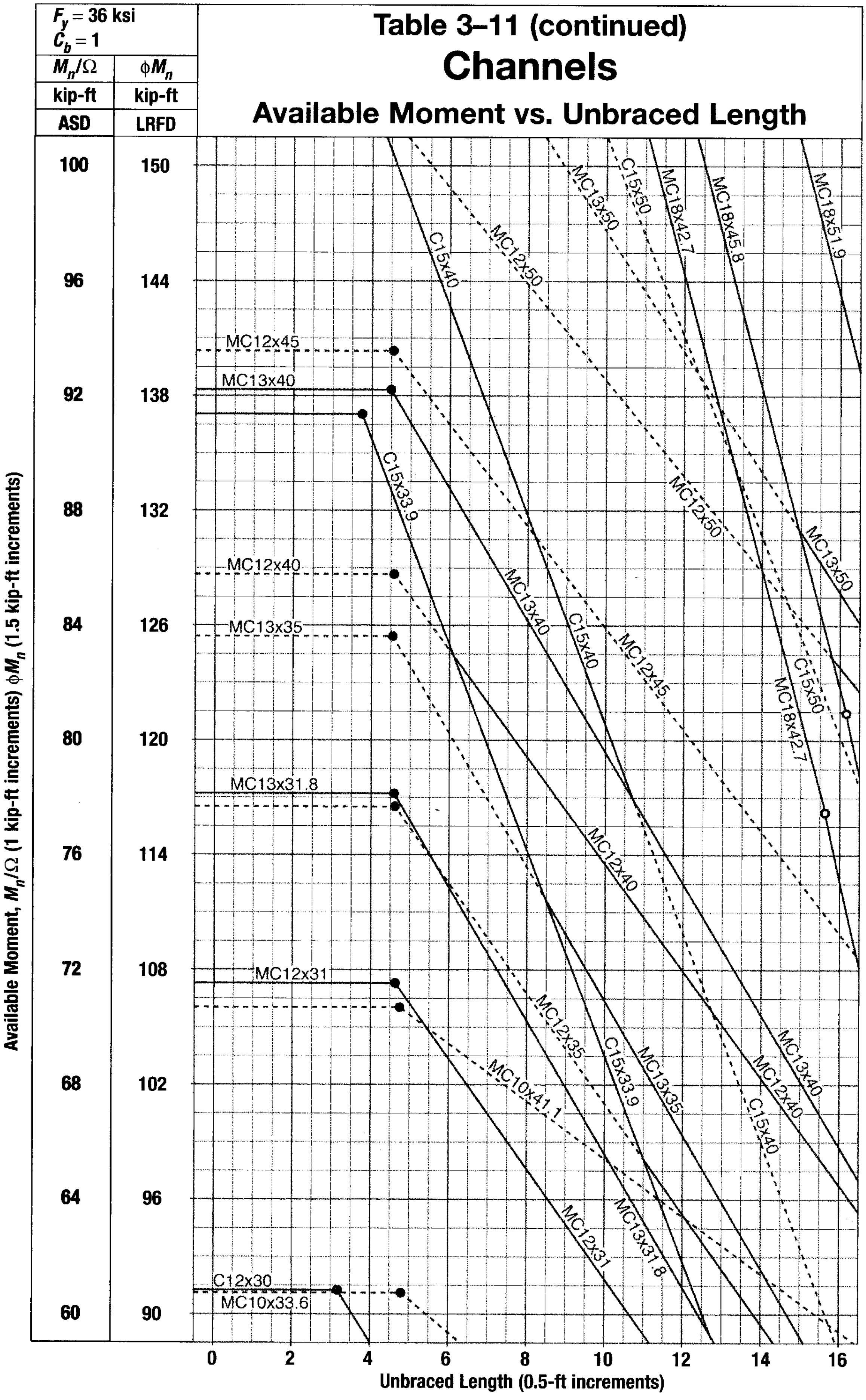


Table 3–11 (continued)
Channels
Available Moment vs. Unbraced Length



$F_y = 36$ ksi
$C_b = 1$

Table 3–11 (continued)
Channels
Available Moment vs. Unbraced Length

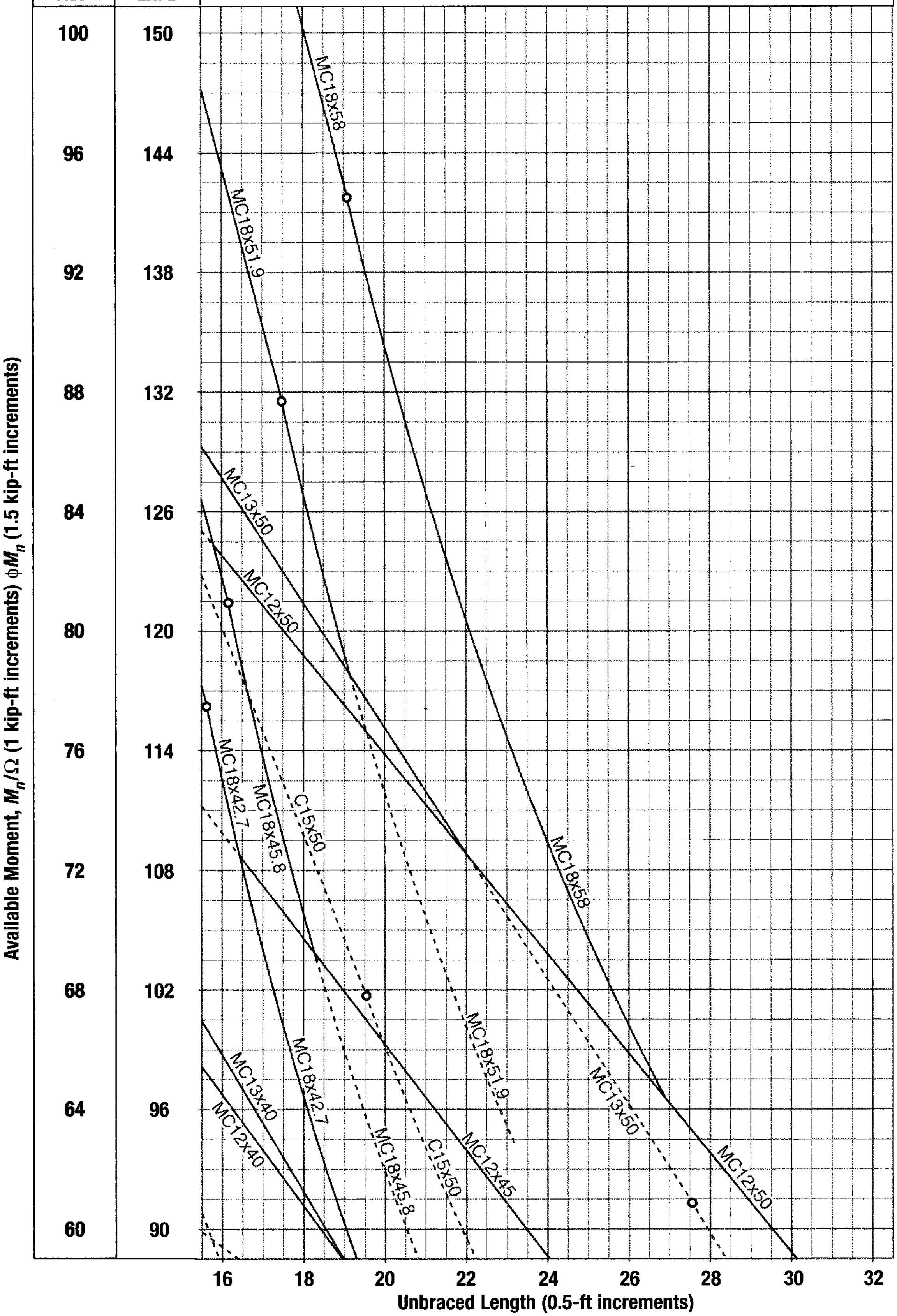
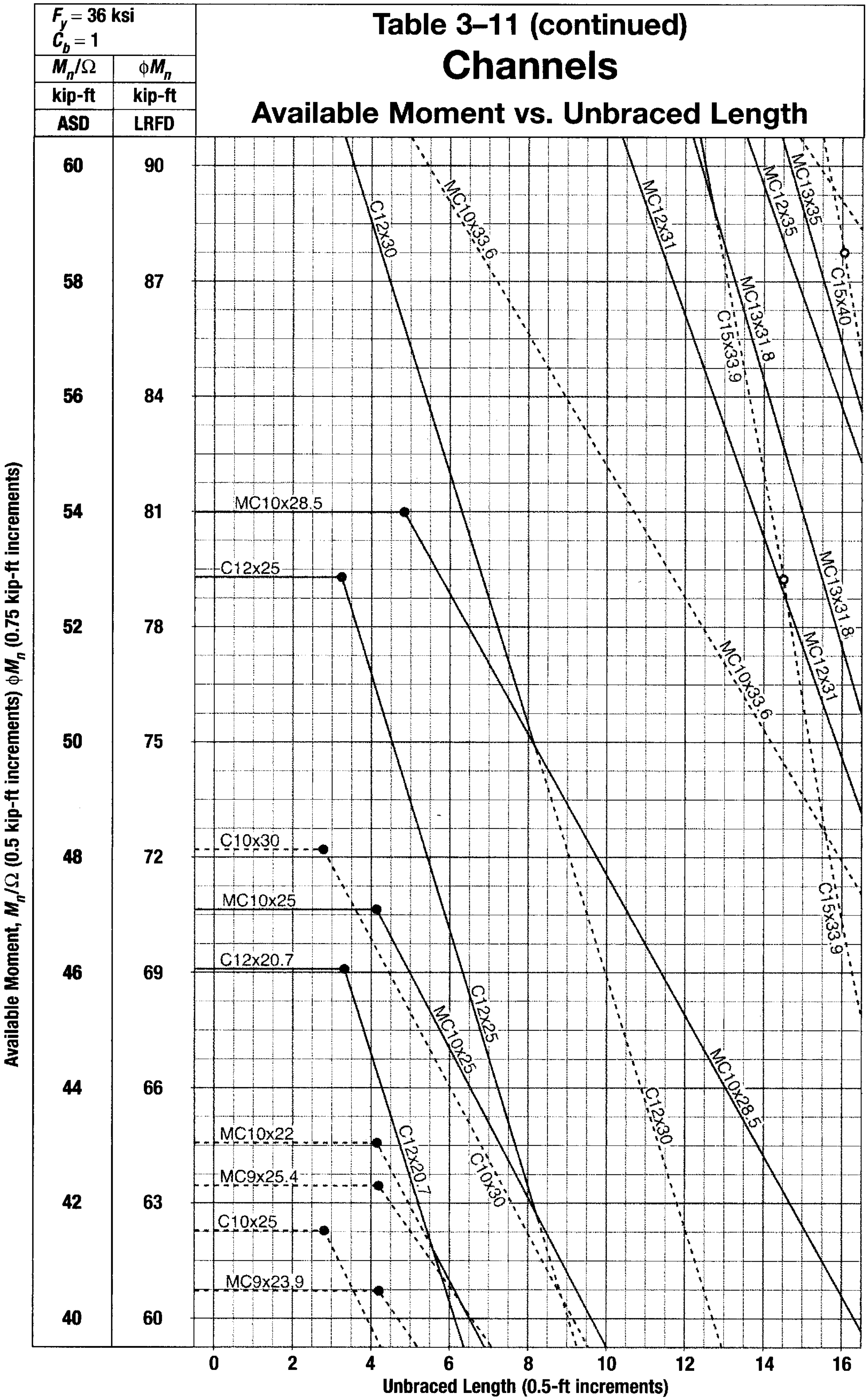


Table 3–11 (continued)
Channels
Available Moment vs. Unbraced Length



$F_y = 36$ ksi
$C_b = 1$

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–11 (continued)
Channels
Available Moment vs. Unbraced Length

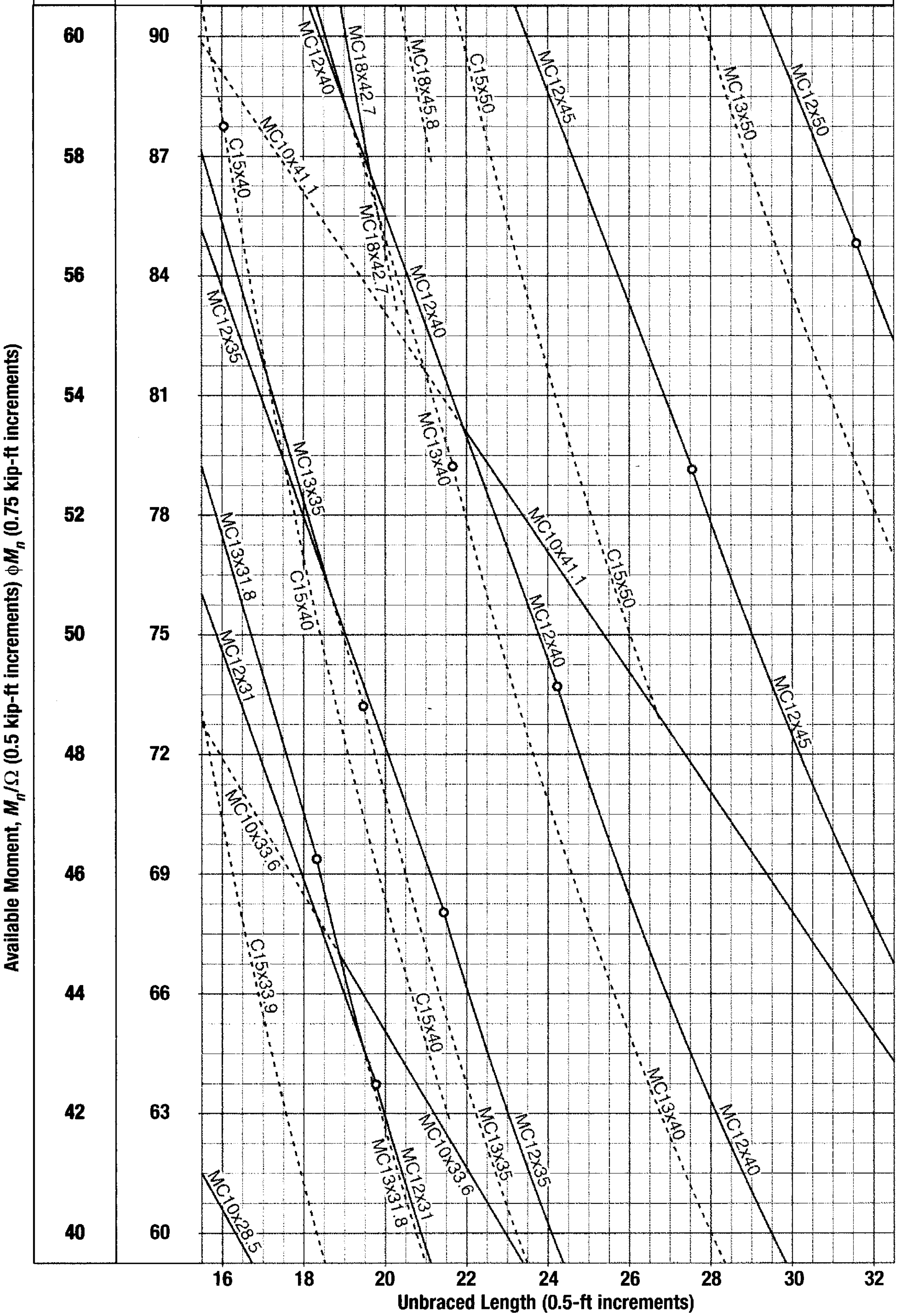
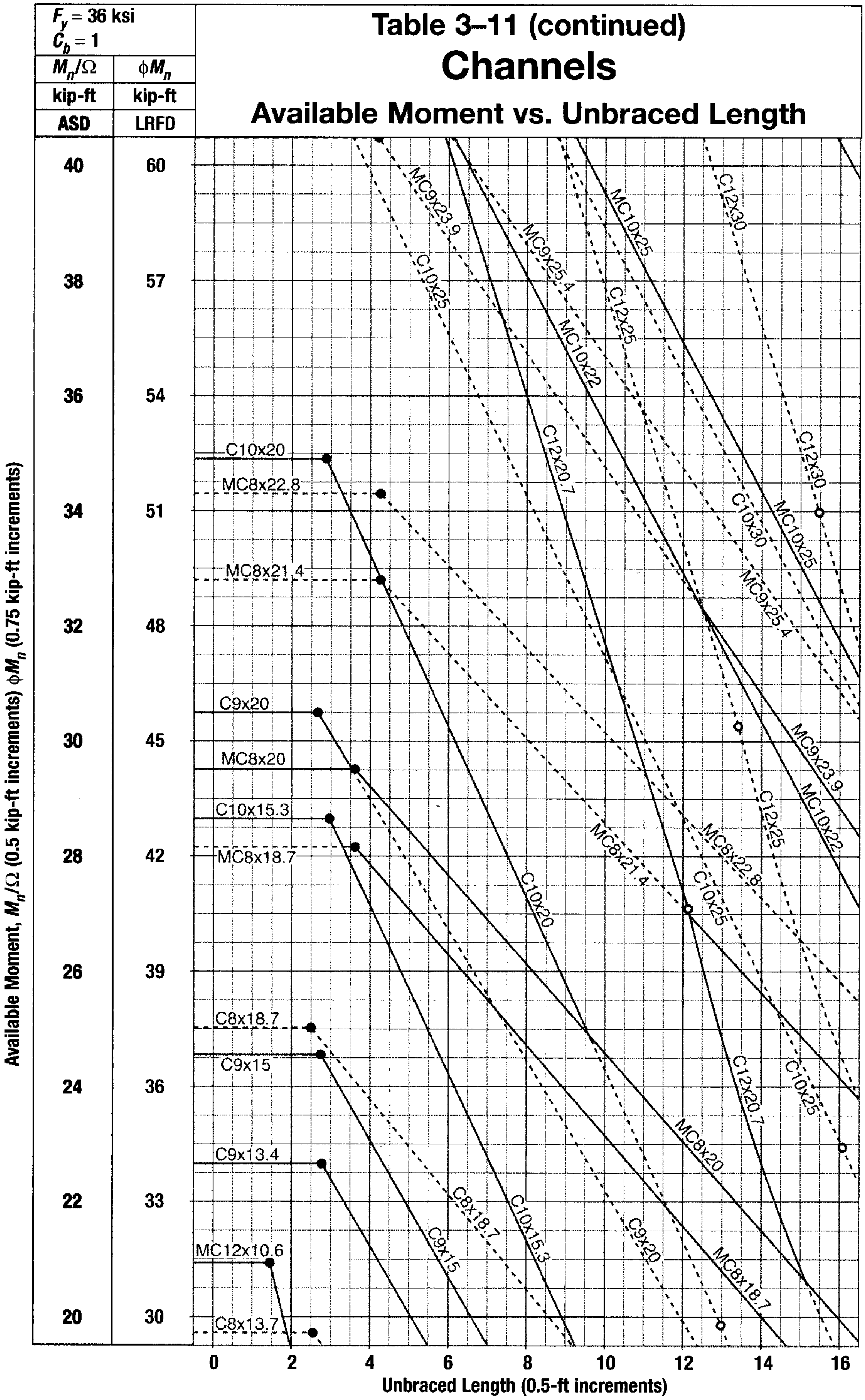


Table 3–11 (continued)
Channels
Available Moment vs. Unbraced Length



F_y = 36 ksi
C_b = 1

M_n/Ω	ϕM_n
kip-ft	kip-ft
ASD	LRFD

Table 3–11 (continued)
Channels
Available Moment vs. Unbraced Length

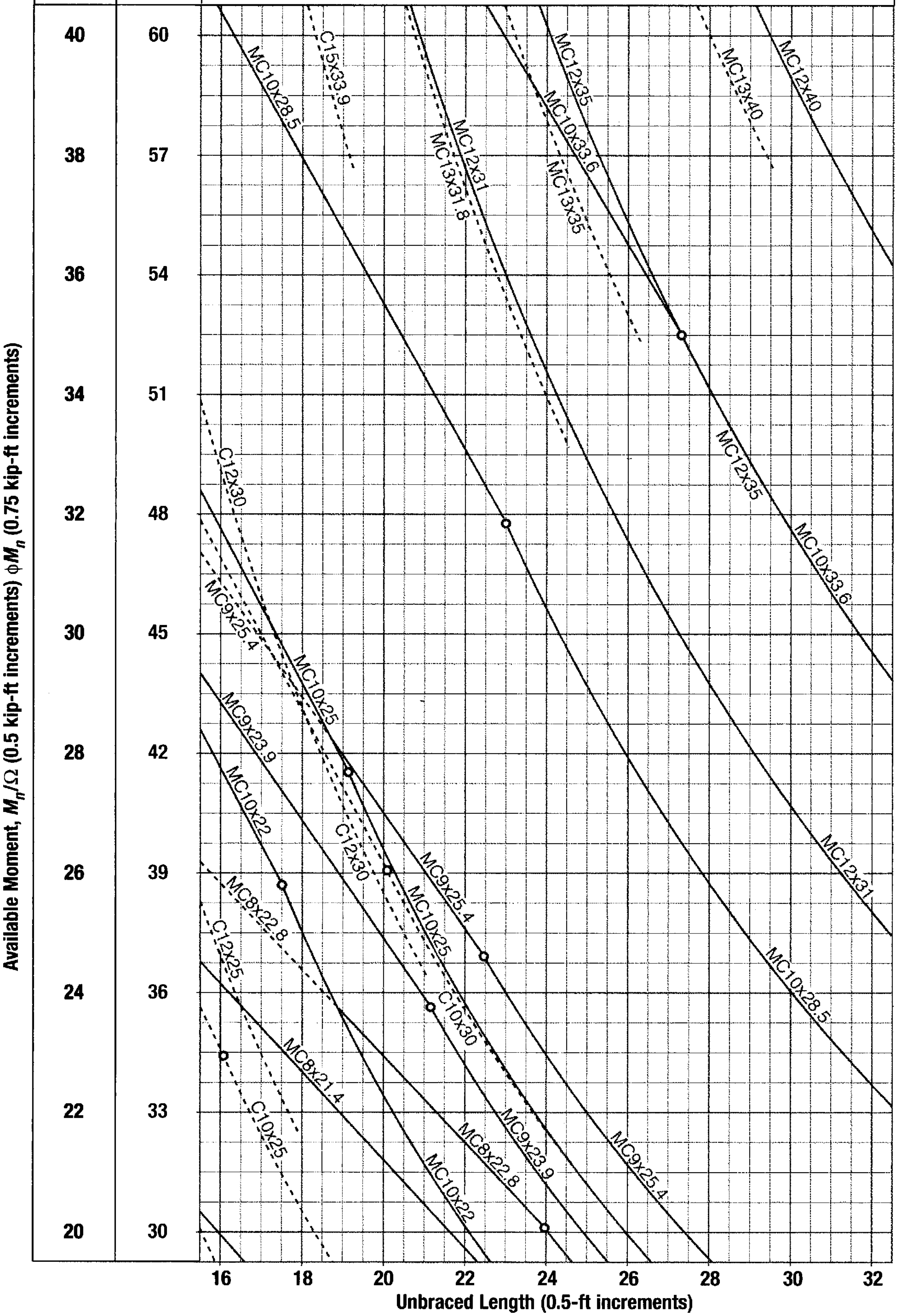
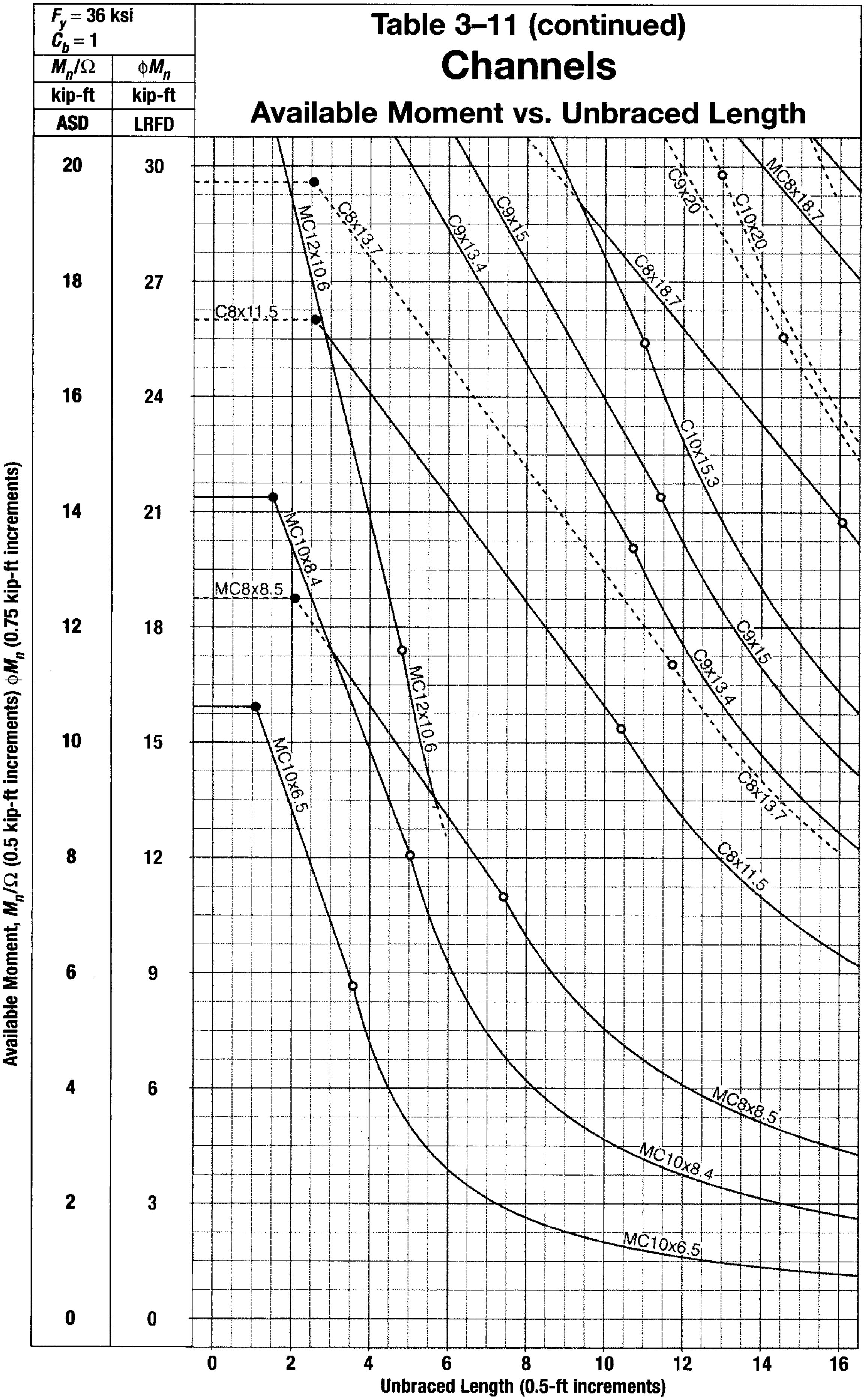
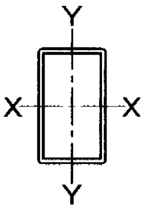


Table 3–11 (continued)
Channels
Available Moment vs. Unbraced Length





HSS20–HSS12

Table 3–12
Available Flexural Strength, kip-ft
Rectangular HSS

F_y = 46 ksi

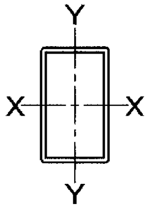
Shape		X-Axis		Y-Axis	
		M_n/Ω	ϕM_n	M_n/Ω	ϕM_n
		ASD	LRFD	ASD	LRFD
HSS20×12×	5/8	**528**	794	**350**	527
	1/2	**432**	649	**254**	382
	3/8	**305**	459	**169**	255
	5/16	**226**	339	**130**	196
HSS20×8×	5/8	**425**	638	**209**	314
	1/2	**349**	524	**152**	229
	3/8	**269**	404	**101**	152
	5/16	**223**	336	**76.8**	115
HSS20×4×	1/2	**264**	397	**62.7**	94.3
	3/8	**205**	308	**42.2**	63.4
	5/16	**171**	257	**32.1**	48.3
	1/4	**131**	198	**22.8**	34.3
HSS18×6×	5/8	**310**	466	**140**	210
	1/2	**257**	386	**102**	153
	3/8	**198**	298	**68.0**	102
	5/16	**168**	252	**52.2**	78.5
	1/4	**132**	198	**37.3**	56.1
HSS16×12×	5/8	**379**	569	**310**	466
	1/2	**310**	466	**240**	360
	3/8	**221**	333	**159**	238
	5/16	**166**	249	**123**	185
HSS16×8×	5/8	**296**	445	**182**	273
	1/2	**243**	366	**142**	213
	3/8	**188**	283	**94.3**	142
	5/16	**159**	240	**73.0**	110
	1/4	**119**	178	**52.6**	79.1
HSS16×4×	5/8	**213**	321	**74.6**	112
	1/2	**177**	267	**58.8**	88.3
	3/8	**138**	208	**39.4**	59.2
	5/16	**117**	176	**30.4**	45.7
	1/4	**94.3**	142	**21.8**	32.8
	3/16	**66.9**	100	**13.9**	20.9
HSS14×10×	5/8	**275**	414	**218**	328
	1/2	**227**	341	**180**	271
	3/8	**175**	263	**120**	180
	5/16	**137**	207	**93.2**	140
	1/4	**97.3**	146	**68.2**	103
HSS14×6×	5/8	**204**	306	**111**	167
	1/2	**169**	254	**92.7**	139
	3/8	**131**	198	**62.6**	94.2
	5/16	**112**	168	**48.7**	73.2
	1/4	**90.9**	137	**35.2**	53.0
	3/16	**62.7**	94.3	**22.8**	34.2
HSS14×4×	5/8	**168**	252	**65.4**	98.3
	1/2	**140**	211	**55.4**	83.3
	3/8	**110**	165	**37.5**	56.3
	5/16	**93.3**	140	**29.2**	43.9
	1/4	**76.2**	115	**21.1**	31.8
	3/16	**55.4**	83.2	**13.6**	20.4
HSS12×10×	1/2	**181**	272	**160**	240
	3/8	**140**	211	**116**	175
	5/16	**111**	166	**88.7**	133
	1/4	**78.9**	119	**65.5**	98.5
HSS12×8×	5/8	**188**	283	**142**	214
	1/2	**156**	235	**118**	178
	3/8	**122**	183	**86.8**	130
	5/16	**103**	155	**66.3**	99.7
	1/4	**77.8**	117	**48.8**	73.4
	3/16	**50.0**	75.2	**32.1**	48.3
HSS12×6×	5/8	**158**	237	**96.6**	145
	1/2	**132**	198	**80.9**	122
	3/8	**103**	155	**59.9**	90.1
	5/16	**87.5**	132	**46.1**	69.4
	1/4	**71.4**	107	**33.8**	50.8
	3/16	**49.6**	74.6	**22.0**	33.1
HSS12×4×	5/8	**127**	192	**56.3**	84.6
	1/2	**107**	161	**47.9**	71.9
	3/8	**84.2**	127	**35.8**	53.8
	5/16	**71.9**	108	**27.7**	41.6
	1/4	**58.8**	88.4	**20.3**	30.5
	3/16	**44.3**	66.6	**13.1**	19.7
HSS12×3½×	3/8	**79.6**	120	**30.2**	45.4
	5/16	**67.9**	102	**23.4**	35.1

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Note: Values above are reduced for compactness criteria, when appropriate. See Table 1–12 for limiting dimensions for compactness.

F_y = 46 ksi

Table 3–12 (continued)
Available Flexural Strength, kip-ft
Rectangular HSS

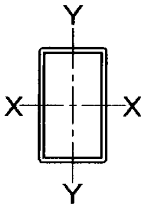


HSS12–HSS8

Shape		X-Axis		Y-Axis	
		M_n/Ω	ϕM_n	M_n/Ω	ϕM_n
		ASD	LRFD	ASD	LRFD
HSS12×3×	5/16	**64.0**	96.2	**19.2**	28.8
	1/4	**52.5**	79.0	**14.1**	21.2
	3/16	**39.6**	59.5	**9.15**	13.7
HSS12×2×	5/16	**56.2**	84.5	**11.2**	16.8
	1/4	**46.3**	69.5	**8.37**	12.6
	3/16	**34.9**	52.4	**5.48**	8.24
HSS10×8×	5/8	**143**	215	**122**	184
	1/2	**119**	179	**102**	153
	3/8	**93.0**	140	**79.8**	120
	5/16	**79.0**	119	**63.8**	95.9
	1/4	**60.0**	90.2	**46.1**	69.2
	3/16	**39.0**	58.6	**30.7**	46.2
HSS10×6×	5/8	**118**	177	**82.1**	123
	1/2	**98.7**	148	**69.1**	104
	3/8	**77.5**	116	**54.4**	81.8
	5/16	**66.1**	99.3	**43.9**	65.9
	1/4	**54.1**	81.3	**31.8**	47.9
	3/16	**37.9**	57.0	**21.1**	31.7
HSS10×5×	3/8	**69.8**	105	**42.9**	64.5
	5/16	**59.6**	89.5	**34.7**	52.2
	1/4	**48.8**	73.4	**25.3**	38.0
	3/16	**37.3**	56.1	**16.7**	25.1
HSS10×4×	5/8	**92.6**	139	**47.2**	70.9
	1/2	**78.3**	118	**40.3**	60.6
	3/8	**62.0**	93.2	**32.2**	48.4
	5/16	**53.1**	79.8	**26.1**	39.3
	1/4	**43.6**	65.5	**19.1**	28.7
	3/16	**33.4**	50.2	**12.6**	18.9
	1/8	**20.7**	31.1	**6.84**	10.3
HSS10×3½	1/2	**73.2**	110	**33.8**	50.8
	3/8	**58.2**	87.4	**27.2**	40.8
	5/16	**49.8**	74.9	**22.1**	33.2
	1/4	**41.0**	61.6	**16.1**	24.3
	3/16	**31.5**	47.3	**10.6**	16.0
	1/8	**20.3**	30.5	**5.75**	8.65
HSS10×3×	3/8	**54.3**	81.6	**22.3**	33.6
	5/16	**46.6**	70.0	**18.1**	27.3
	1/4	**38.4**	57.7	**13.3**	20.0
	3/16	**29.5**	44.3	**8.75**	13.2
	1/8	**19.0**	28.5	**4.72**	7.10
HSS10×2×	3/8	**46.6**	70.0	**13.2**	19.9
	5/16	**40.1**	60.3	**10.8**	16.2
	1/4	**33.2**	49.8	**7.86**	11.8
	3/16	**25.6**	38.4	**5.25**	7.89
	1/8	**16.3**	24.6	**2.83**	4.25
HSS9×7×	5/8	**111**	167	**93.0**	140
	1/2	**92.9**	140	**78.1**	117
	3/8	**72.9**	110	**61.4**	92.3
	5/16	**62.2**	93.4	**52.4**	78.7
	1/4	**50.9**	76.5	**37.3**	56.0
	3/16	**32.3**	48.6	**25.0**	37.6
HSS9×5×	5/8	**88.3**	133	**58.1**	87.3
	1/2	**74.7**	112	**49.3**	74.1
	3/8	**59.1**	88.8	**39.2**	58.9
	5/16	**50.5**	75.9	**33.6**	50.5
	1/4	**41.5**	62.4	**24.3**	36.5
	3/16	**31.8**	47.8	**16.2**	24.3
HSS9×3×	1/2	**56.4**	84.8	**24.8**	37.3
	3/8	**45.2**	67.9	**20.2**	30.4
	5/16	**38.9**	58.5	**17.5**	26.3
	1/4	**32.1**	48.3	**12.7**	19.1
	3/16	**24.7**	37.2	**8.50**	12.8
HSS8×6×	5/8	**82.8**	124	**67.7**	102
	1/2	**69.9**	105	**57.3**	86.1
	3/8	**55.3**	83.1	**45.4**	68.2
	5/16	**47.3**	71.1	**38.8**	58.4
	1/4	**38.8**	58.4	**30.1**	45.2
	3/16	**27.5**	41.4	**19.7**	29.7

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Note: Values above are reduced for compactness criteria, when appropriate. See Table 1–12 for limiting dimensions for compactness.



HSS8–HSS5

Table 3–12 (continued)
Available Flexural Strength, kip-ft
Rectangular HSS

F_y = 46 ksi

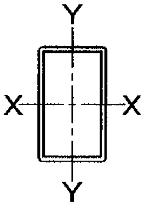
Shape		X-Axis		Y-Axis	
		M_n/Ω	ϕM_n	M_n/Ω	ϕM_n
		ASD	LRFD	ASD	LRFD
HSS8×4×	5/8	63.0	94.7	38.1	57.2
	1/2	53.8	80.9	32.8	49.3
	3/8	43.0	64.7	26.4	39.6
	5/16	37.0	55.6	22.7	34.2
	1/4	30.5	45.9	17.8	26.7
	3/16	23.5	35.3	11.8	17.7
	1/8	14.7	22.1	6.53	9.82
HSS8×3×	1/2	45.8	68.8	22.1	33.3
	3/8	36.9	55.5	18.1	27.2
	5/16	31.9	47.9	15.7	23.6
	1/4	26.4	39.6	12.3	18.6
	3/16	20.4	30.6	8.19	12.3
	1/8	13.8	20.8	4.52	6.79
HSS8×2×	3/8	30.8	46.3	10.6	15.9
	5/16	26.7	40.1	9.33	14.0
	1/4	22.2	33.4	7.37	11.1
	3/16	17.2	25.9	4.90	7.37
	1/8	11.7	17.6	2.71	4.07
HSS7×5×	1/2	50.2	75.4	39.6	59.6
	3/8	40.1	60.2	31.7	47.7
	5/16	34.4	51.8	27.3	41.1
	1/4	28.4	42.7	22.6	33.9
	3/16	21.8	32.8	14.9	22.4
	1/8	12.1	18.2	8.47	12.7
HSS7×4×	1/2	43.2	64.9	29.0	43.6
	3/8	34.7	52.2	23.4	35.2
	5/16	30.0	45.0	20.3	30.5
	1/4	24.8	37.3	16.8	25.3
	3/16	19.1	28.7	11.2	16.8
	1/8	12.1	18.1	6.33	9.51
HSS7×3×	1/2	36.2	54.4	19.4	29.2
	3/8	29.4	44.2	16.0	24.0
	5/16	25.5	38.3	13.9	20.9
	1/4	21.2	31.8	11.6	17.4
	3/16	16.4	24.6	7.80	11.7
	1/8	11.3	17.0	4.38	6.58
HSS7×2×	1/4	17.5	26.4	6.94	10.4
	3/16	13.7	20.5	4.67	7.01
	1/8	9.49	14.3	2.63	3.95
HSS6×5×	1/2	39.5	59.4	34.8	52.3
	3/8	31.8	47.8	28.0	42.1
	5/16	27.4	41.2	24.2	36.3
	1/4	22.7	34.1	20.0	30.1
	3/16	17.5	26.3	14.5	21.8
	1/8	9.80	14.7	8.12	12.2
HSS6×4×	1/2	33.6	50.5	25.2	37.9
	3/8	27.3	41.0	20.5	30.8
	5/16	23.6	35.4	17.8	26.7
	1/4	19.6	29.4	14.8	22.2
	3/16	15.2	22.8	10.8	16.2
	1/8	9.65	14.5	6.07	9.12
HSS6×3×	1/2	27.7	41.7	16.7	25.1
	3/8	22.7	34.2	13.8	20.8
	5/16	19.8	29.7	12.1	18.2
	1/4	16.5	24.8	10.1	15.2
	3/16	12.8	19.3	7.46	11.2
	1/8	8.89	13.4	4.20	6.31
HSS6×2×	3/8	18.2	27.4	7.94	11.9
	5/16	16.0	24.0	7.05	10.6
	1/4	13.4	20.2	5.99	9.01
	3/16	10.5	15.8	4.46	6.70
	1/8	7.33	11.0	2.52	3.79
HSS5×4×	1/2	25.1	37.8	21.5	32.2
	3/8	20.6	30.9	17.6	26.5
	5/16	17.9	26.9	15.3	23.0
	1/4	14.9	22.4	12.8	19.2
	3/16	11.6	17.4	9.95	15.0
	1/8	7.45	11.2	5.72	8.60

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Note: Values above are reduced for compactness criteria, when appropriate. See Table 1–12 for limiting dimensions for compactness.

F_y = 46 ksi

Table 3–12 (continued)
Available Flexural Strength, kip-ft
Rectangular HSS



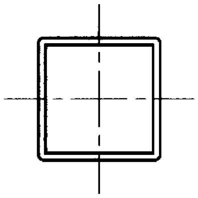
HSS5–HSS2

Shape		X-Axis		Y-Axis	
		M_n/Ω	ϕM_n	M_n/Ω	ϕM_n
		ASD	LRFD	ASD	LRFD
HSS5×3×	1/2	20.3	30.5	14.0	21.1
	3/8	16.8	25.3	11.7	17.6
	5/16	14.7	22.1	10.3	15.4
	1/4	12.4	18.6	8.65	13.0
	3/16	9.66	14.5	6.79	10.2
	1/8	6.73	10.1	3.96	5.95
HSS5×2½×	1/4	11.1	16.7	6.78	10.2
	3/16	8.70	13.1	5.35	8.04
	1/8	6.08	9.14	3.14	4.72
HSS5×2×	3/8	13.1	19.7	6.62	9.95
	5/16	11.6	17.4	5.91	8.88
	1/4	9.81	14.7	5.05	7.59
	3/16	7.74	11.6	4.02	6.04
	1/8	5.43	8.16	2.37	3.57
HSS4×3×	3/8	11.7	17.7	9.58	14.4
	5/16	10.4	15.6	8.47	12.7
	1/4	8.76	13.2	7.17	10.8
	3/16	6.90	10.4	5.66	8.50
	1/8	4.84	7.27	3.73	5.61
HSS4×2½×	3/8	10.3	15.5	7.34	11.0
	5/16	9.12	13.7	6.53	9.82
	1/4	7.75	11.6	5.57	8.37
	3/16	6.13	9.22	4.42	6.65
	1/8	4.32	6.49	2.94	4.42
HSS4×2×	3/8	8.82	13.3	5.30	7.96
	5/16	7.88	11.8	4.76	7.16
	1/4	6.74	10.1	4.10	6.17
	3/16	5.37	8.07	3.29	4.94
	1/8	3.80	5.71	2.21	3.32
HSS3½×2½×	3/8	8.24	12.4	6.48	9.74
	5/16	7.35	11.1	5.79	8.71
	1/4	6.28	9.44	4.96	7.46
	3/16	5.00	7.51	3.96	5.95
	1/8	3.54	5.32	2.81	4.22

Shape		X-Axis		Y-Axis	
		M_n/Ω	ϕM_n	M_n/Ω	ϕM_n
		ASD	LRFD	ASD	LRFD
HSS3½×2×	1/4	5.41	8.13	3.63	5.46
	3/16	4.33	6.51	2.92	4.40
	1/8	3.09	4.64	2.09	3.15
HSS3½×1½×	1/4	4.53	6.82	2.43	3.65
	3/16	3.67	5.51	1.99	2.99
	1/8	2.64	3.96	1.45	2.17
HSS3×2½×	5/16	5.75	8.65	5.06	7.60
	1/4	4.95	7.44	4.36	6.55
	3/16	3.96	5.96	3.49	5.25
	1/8	2.82	4.24	2.49	3.74
HSS3×2×	5/16	4.85	7.29	3.62	5.45
	1/4	4.21	6.33	3.16	4.75
	3/16	3.40	5.11	2.56	3.85
	1/8	2.44	3.66	1.84	2.77
HSS3×1½×	1/4	3.47	5.21	2.09	3.14
	3/16	2.83	4.26	1.73	2.59
	1/8	2.05	3.09	1.26	1.90
HSS3×1×	3/16	2.27	3.41	0.991	1.49
	1/8	1.67	2.51	0.747	1.12
HSS2½×2×	1/4	3.14	4.73	2.69	4.04
	3/16	2.56	3.86	2.20	3.30
	1/8	1.86	2.79	1.59	2.39
HSS2½×1½×	1/4	2.54	3.81	1.75	2.64
	3/16	2.10	3.16	1.46	2.20
	1/8	1.54	2.31	1.08	1.62
HSS2½×1×	3/16	1.64	2.46	0.826	1.24
	1/8	1.22	1.84	0.629	0.945
HSS2¼×2×	3/16	2.19	3.28	2.01	3.03
	1/8	1.59	2.39	1.47	2.20
HSS2×1½×	3/16	1.47	2.20	1.20	1.80
	1/8	1.09	1.64	0.893	1.34
HSS2×1×	3/16	1.10	1.66	0.661	0.994
	1/8	0.840	1.26	0.511	0.768

ASD	LRFD
Ω_b = 1.67	ϕ_b = 0.90

Note: Values above are reduced for compactness criteria, when appropriate. See Table 1-12 for limiting dimensions for compactness.



HSS16–HSS4½

Table 3-13

Available Flexural Strength, kip-ft

Square HSS

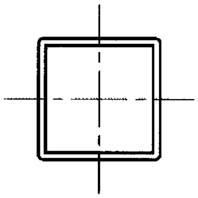
F_y = 46 ksi

Shape		M_n/Ω	ϕM_n
		ASD	LRFD
HSS16×16×	5/8	**459**	690
	1/2[f]	**352**	529
	3/8[f]	**242**	363
	5/16[f]	**187**	281
HSS14×14×	5/8	**347**	521
	1/2	**285**	428
	3/8[f]	**189**	284
	5/16[f]	**151**	227
HSS12×12×	5/8	**250**	376
	1/2	**206**	309
	3/8[f]	**149**	223
	5/16[f]	**116**	175
	1/4[f]	**86.3**	130
	3/16[f]	**56.9**	85.6
HSS10×10×	5/8	**168**	252
	1/2	**139**	210
	3/8	**108**	163
	5/16[f]	**86.1**	129
	1/4[f]	**64.4**	96.8
	3/16[f]	**42.8**	64.3
HSS9×9×	5/8	**133**	200
	1/2	**111**	167
	3/8	**86.8**	130
	5/16	**73.8**	111
	1/4[f]	**52.0**	78.2
	3/16[f]	**36.3**	54.6
	1/8[f]	**20.3**	30.5
HSS8×8×	5/8	**103**	154
	1/2	**86.0**	129
	3/8	**67.6**	102
	5/16	**57.6**	86.6
	1/4[f]	**44.1**	66.3
	3/16[f]	**30.1**	45.3
	1/8[f]	**16.9**	25.4
HSS7×7×	5/8	**75.9**	114
	1/2	**64.1**	96.4
	3/8	**50.7**	76.2
	5/16	**43.4**	65.2
	1/4	**35.6**	53.6
	3/16[f]	**23.6**	35.5
	1/8[f]	**13.7**	20.6
HSS6×6×	5/8	**53.2**	80.0
	1/2	**45.4**	68.3
	3/8	**36.3**	54.6
	5/16	**31.2**	46.9
	1/4	**25.7**	38.7
	3/16[f]	**18.5**	27.8
	1/8[f]	**10.8**	16.2
HSS5½×5½×	3/8	**30.0**	45.1
	5/16	**25.9**	38.9
	1/4	**21.4**	32.2
	3/16[f]	**16.4**	24.6
	1/8[f]	**9.37**	14.1
HSS5×5×	1/2	**30.0**	45.0
	3/8	**24.3**	36.5
	5/16	**21.0**	31.6
	1/4	**17.5**	26.2
	3/16	**13.5**	20.3
	1/8[f]	**8.04**	12.1
HSS4½×4½×	1/2	**23.4**	35.2
	3/8	**19.2**	28.8
	5/16	**16.7**	25.1
	1/4	**13.9**	20.9
	3/16	**10.8**	16.3
	1/8[f]	**6.48**	9.73

[f] Shape exceeds compact limit for flexure with F_y = 46 ksi.

F_y = 46 ksi

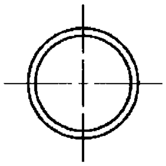
Table 3–13 (continued)
Available Flexural Strength, kip-ft
Square HSS



HSS4–HSS2

Shape		M_n/Ω	ϕM_n
		ASD	LRFD
HSS4×4×	1/2	17.7	26.6
	3/8	14.7	22.1
	5/16	12.8	19.3
	1/4	10.8	16.2
	3/16	8.42	12.7
	1/8[f]	5.48	8.23
HSS3½×3½×	3/8	10.8	16.2
	5/16	9.50	14.3
	1/4	8.03	12.1
	3/16	6.33	9.51
	1/8	4.44	6.67
HSS3×3×	3/8	7.46	11.2
	5/16	6.66	10.0
	1/4	5.69	8.55
	3/16	4.53	6.81
	1/8	3.21	4.82
HSS2½×2½×	5/16	4.32	6.49
	1/4	3.75	5.64
	3/16	3.03	4.55
	1/8	2.17	3.27
HSS2¼×2¼×	1/4	2.93	4.41
	3/16	2.39	3.60
	1/8	1.73	2.60
HSS2×2×	1/4	2.21	3.33
	3/16	1.83	2.75
	1/8	1.34	2.02

[f] Shape exceeds compact limit for flexure with F_y = 46 ksi.



HSS20.000–HSS6.625

Table 3–14
Available Flexural Strength, kip-ft
Round HSS

$F_y = 42$ ksi

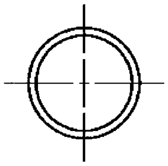
Shape		M_n/Ω	ϕM_n
		ASD	LRFD
HSS20.000×	0.500	371	558
	0.375[f]	273	410
HSS18.000×	0.500	300	450
	0.375[f]	225	338
HSS16.000×	0.625	289	435
	0.500	235	353
	0.438	207	312
	0.375	179	269
	0.312[f]	147	221
	0.250[f]	114	171
HSS14.000×	0.625	220	331
	0.500	179	268
	0.375	136	205
	0.312	115	172
	0.250[f]	88.8	133
HSS12.750×	0.500	147	221
	0.375	113	169
	0.250[f]	74.6	112
HSS10.750×	0.500	103	155
	0.375	79.2	119
	0.250	54.0	81.2
HSS10.000×	0.625	108	163
	0.500	88.7	133
	0.375	68.2	102
	0.312	57.5	86.4
	0.250	46.6	70.0
	0.188[f]	34.0	51.2
HSS9.625×	0.500	81.8	123
	0.375	63.0	94.6
	0.312	53.2	79.9
	0.250	43.1	64.8
	0.188[f]	31.7	47.7
HSS8.625×	0.625	78.9	119
	0.500	65.0	97.6
	0.375	50.1	75.3
	0.322	43.6	65.5
	0.250	34.4	51.7
	0.188[f]	25.9	39.0
HSS7.625×	0.375	38.8	58.2
	0.328	34.3	51.5
HSS7.500×	0.500	48.3	72.6
	0.375	37.4	56.3
	0.312	31.7	47.7
	0.250	25.8	38.8
	0.188	19.6	29.4
HSS7.000×	0.500	41.7	62.7
	0.375	32.4	48.7
	0.312	27.5	41.3
	0.250	22.4	33.6
	0.188	17.0	25.5
	0.125[f]	11.0	16.6
HSS6.875×	0.500	40.1	60.3
	0.375	31.2	46.9
	0.312	26.5	39.8
	0.250	21.6	32.4
	0.188	16.4	24.6
HSS6.625×	0.500	37.1	55.7
	0.432	32.7	49.1
	0.375	28.8	43.3
	0.312	24.5	36.8
	0.280	22.1	33.2
	0.250	20.0	30.0
	0.188	15.2	22.8
	0.125[f]	9.97	15.0

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[f] Shape exceeds compact limit for flexure with $F_y = 42$ ksi.

F_y = 42 ksi

Table 3–14 (continued)
Available Flexural Strength, kip-ft
Round HSS



HSS6.000–HSS1.660

Shape		M_n/Ω	ϕM_n
		ASD	LRFD
HSS6.000×	0.500	**29.9**	45.0
	0.375	**23.4**	35.2
	0.312	**19.9**	29.9
	0.280	**18.0**	27.0
	0.250	**16.2**	24.4
	0.188	**12.4**	18.6
	0.125[f]	**8.30**	12.5
HSS5.563×	0.500	**25.4**	38.2
	0.375	**19.9**	29.9
	0.258	**14.3**	21.4
	0.188	**10.6**	15.9
	0.134	**7.69**	11.6
HSS5.500×	0.500	**24.8**	37.2
	0.375	**19.4**	29.2
	0.258	**13.9**	20.9
HSS5.000×	0.500	**20.1**	30.2
	0.375	**15.9**	23.8
	0.312	**13.5**	20.4
	0.258	**11.4**	17.1
	0.250	**11.1**	16.7
	0.188	**8.50**	12.8
	0.125	**5.80**	8.72
HSS4.500×	0.375	**12.6**	19.0
	0.337	**11.5**	17.3
	0.237	**8.45**	12.7
	0.188	**6.83**	10.3
	0.125	**4.67**	7.02
HSS4.000×	0.313	**8.41**	12.6
	0.250	**6.94**	10.4
	0.237	**6.60**	9.91
	0.226	**6.33**	9.51
	0.220	**6.19**	9.31
	0.188	**5.34**	8.03
	0.125	**3.67**	5.51
HSS3.500×	0.313	**6.30**	9.47
	0.300	**6.08**	9.14
	0.250	**5.22**	7.85
	0.216	**4.59**	6.90
	0.203	**4.35**	6.53
	0.188	**4.04**	6.07
	0.125	**2.79**	4.19
HSS3.000×	0.250	**3.75**	5.63
	0.216	**3.31**	4.97
	0.203	**3.13**	4.71
	0.188	**2.92**	4.38
	0.152	**2.42**	3.63
	0.134	**2.15**	3.23
	0.125	**2.02**	3.04
HSS2.875×	0.250	**3.42**	5.14
	0.203	**2.86**	4.30
	0.188	**2.66**	4.00
	0.125	**1.85**	2.78
HSS2.500×	0.250	**2.52**	3.79
	0.188	**1.98**	2.97
	0.125	**1.38**	2.08
HSS2.375×	0.250	**2.25**	3.38
	0.218	**2.01**	3.03
	0.188	**1.77**	2.66
	0.154	**1.50**	2.25
	0.125	**1.24**	1.87
HSS1.900×	0.188	**1.09**	1.64
	0.145	**0.883**	1.33
	0.120	**0.746**	1.12
HSS1.660×	0.140	**0.639**	0.961

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[f] Shape exceeds compact limit for flexure with F_y = 42 ksi.

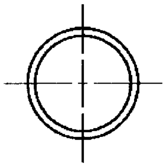


Table 3–15
Pipe
Available Flexural Strength, kip-ft

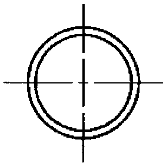
F_y = 35 ksi

Shape	ASD	LRFD
Pipe 12 X-Strong	**123**	184
Pipe 12 Std	**93.8**	141
Pipe 10 X-Strong	**86.0**	129
Pipe 10 Std	**64.4**	96.8
Pipe 8 XX-Strong	**87.2**	131
Pipe 8 X-Strong	**54.1**	81.4
Pipe 8 Std	**36.3**	54.6
Pipe 6 XX-Strong	**47.9**	72.0
Pipe 6 X-Strong	**27.3**	41.0
Pipe 6 Std	**18.5**	27.8
Pipe 5 XX-Strong	**29.1**	43.7
Pipe 5 X-Strong	**16.6**	24.9
Pipe 5 Std	**11.9**	17.9
Pipe 4 XX-Strong	**16.6**	24.9
Pipe 4 X-Strong	**9.65**	14.5
Pipe 4 Std	**7.07**	10.6
Pipe $3\frac{1}{2}$ X-Strong	**7.11**	10.7
Pipe $3\frac{1}{2}$ Std	**5.30**	7.96

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

F_y = 35 ksi

Table 3–15 (continued)
Pipe
Available Flexural Strength, kip-ft



Shape	ASD	LRFD	Shape	ASD	LRFD
Pipe 3 XX-Strong	**8.55**	12.8	Pipe 1¼ X-Strong	**0.686**	1.03
Pipe 3 X-Strong	**5.08**	7.64	Pipe 1¼ Std	**0.533**	0.801
Pipe 3 Std	**3.83**	5.75	Pipe 1 X-Strong	**0.385**	0.579
Pipe 2½ XX-Strong	**5.08**	7.64	Pipe 1 Std	**0.308**	0.463
Pipe 2½ X-Strong	**3.09**	4.64	Pipe ¾ X-Strong	**0.207**	0.311
Pipe 2½ Std	**2.39**	3.59	Pipe ¾ Std	**0.164**	0.247
Pipe 2 XX-Strong	**2.79**	4.19	Pipe ½ X-Strong	**0.120**	0.180
Pipe 2 X-Strong	**1.68**	2.53	Pipe ½ Std	**0.0969**	0.146
Pipe 2 Std	**1.25**	1.87			
Pipe 1½ X-Strong	**0.958**	1.44			
Pipe 1½ Std	**0.736**	1.11			

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

Table 3–16a

Available Shear Stress, ksi

Tension Field Action NOT Included

F_y = 36 ksi

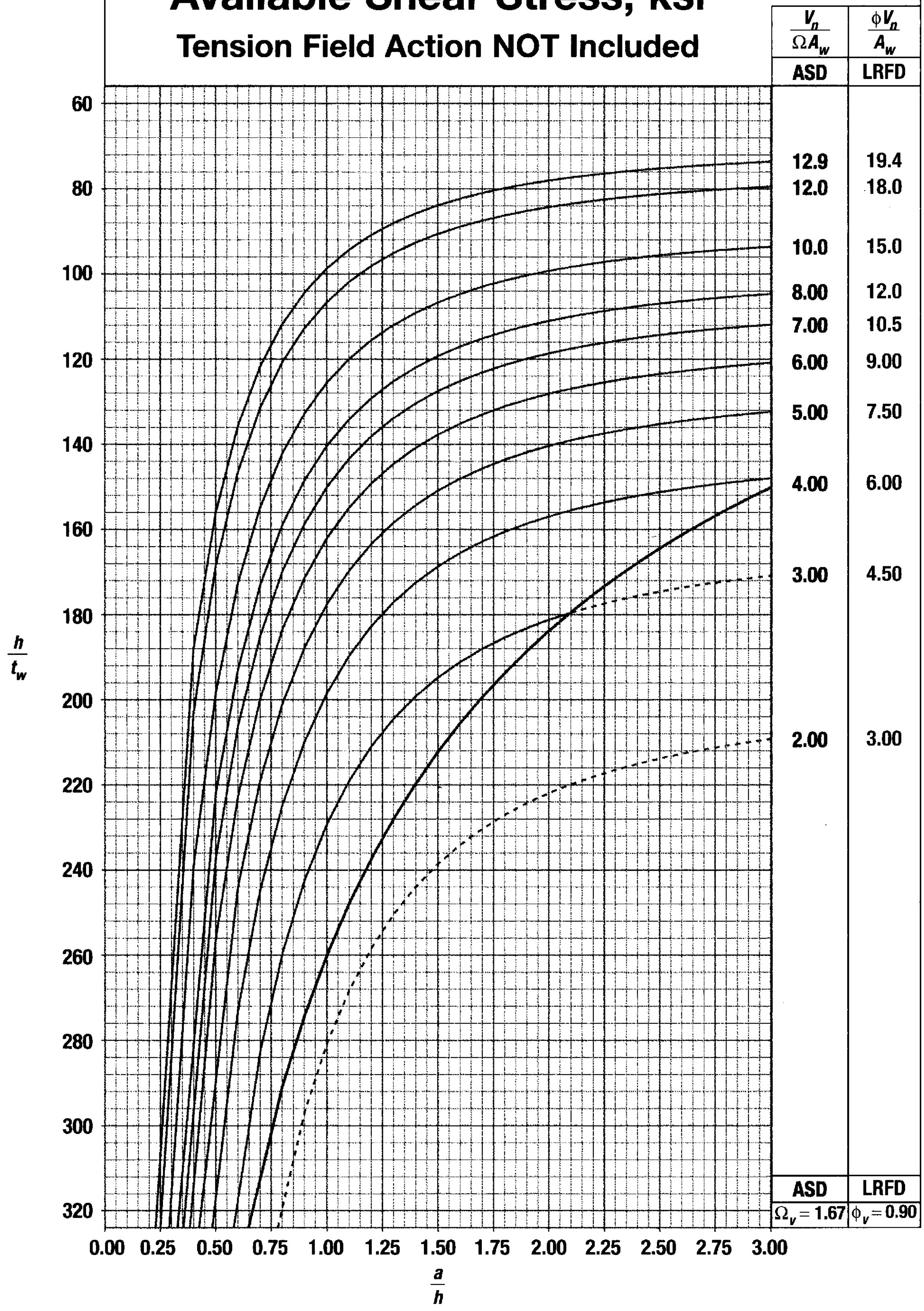


Table 3–16b

Available Shear Stress, ksi

Tension Field Action Included

F_y = 36 ksi

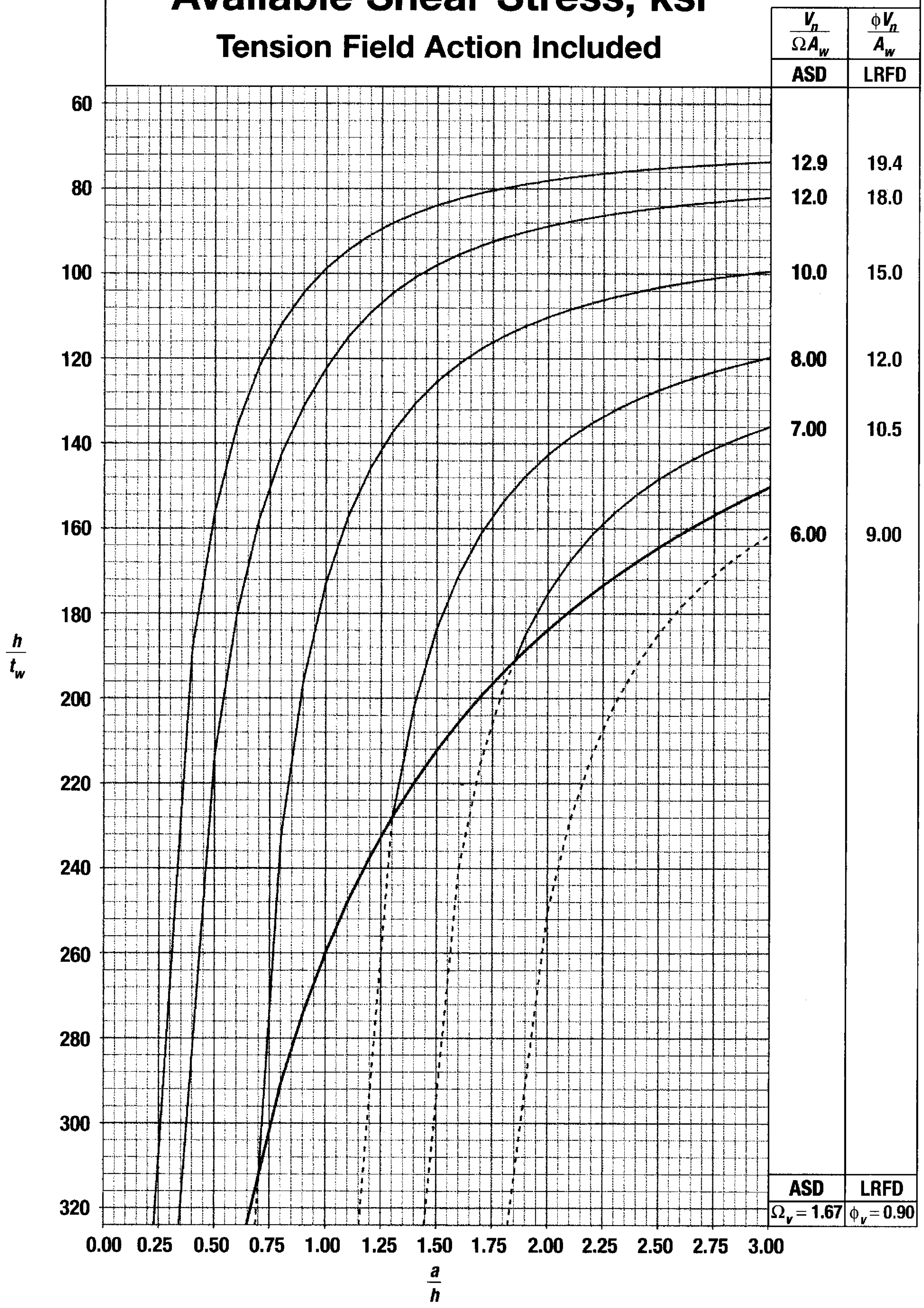


Table 3–17a

Available Shear Stress, ksi

Tension Field Action NOT Included

F_y = 50 ksi

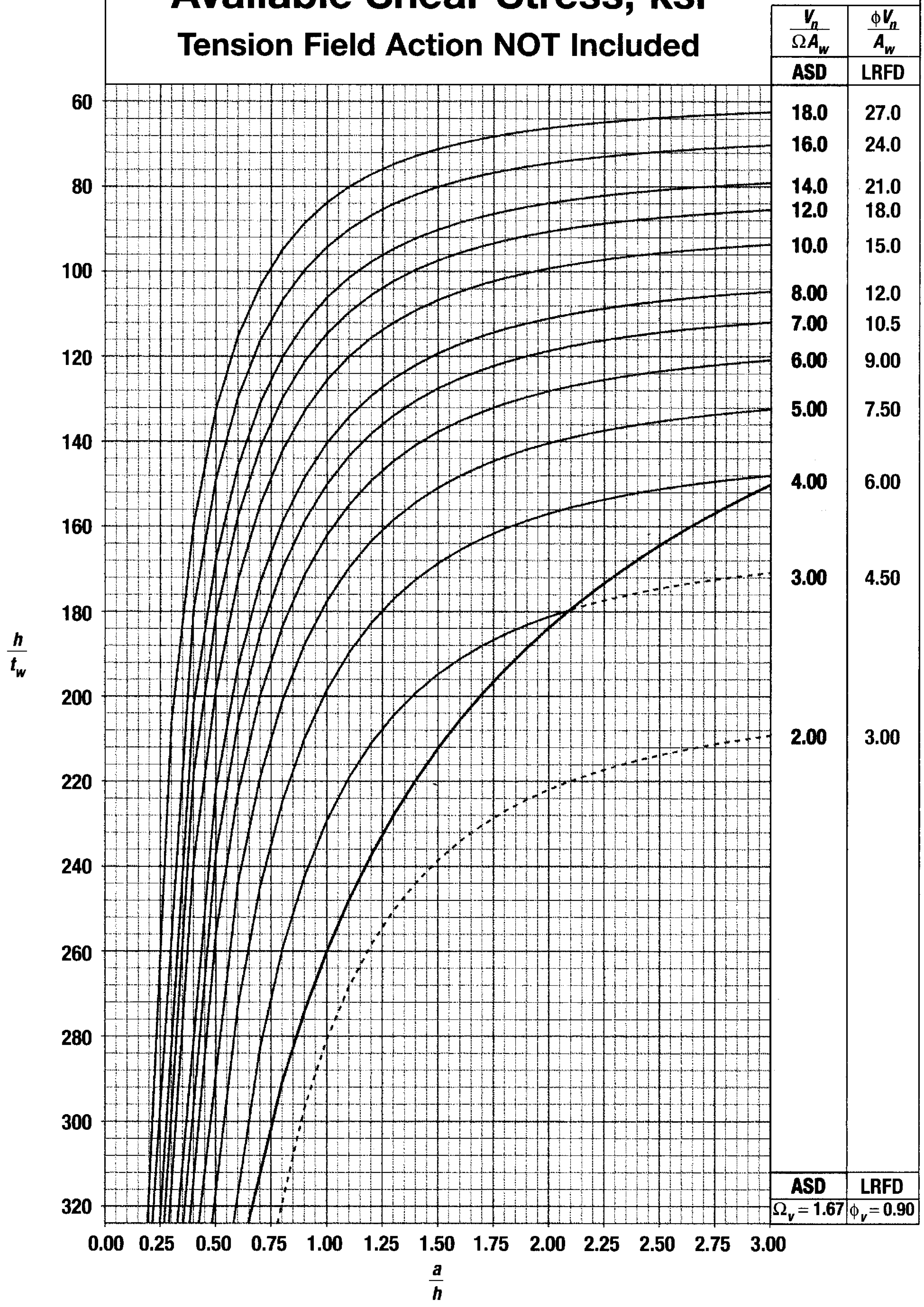
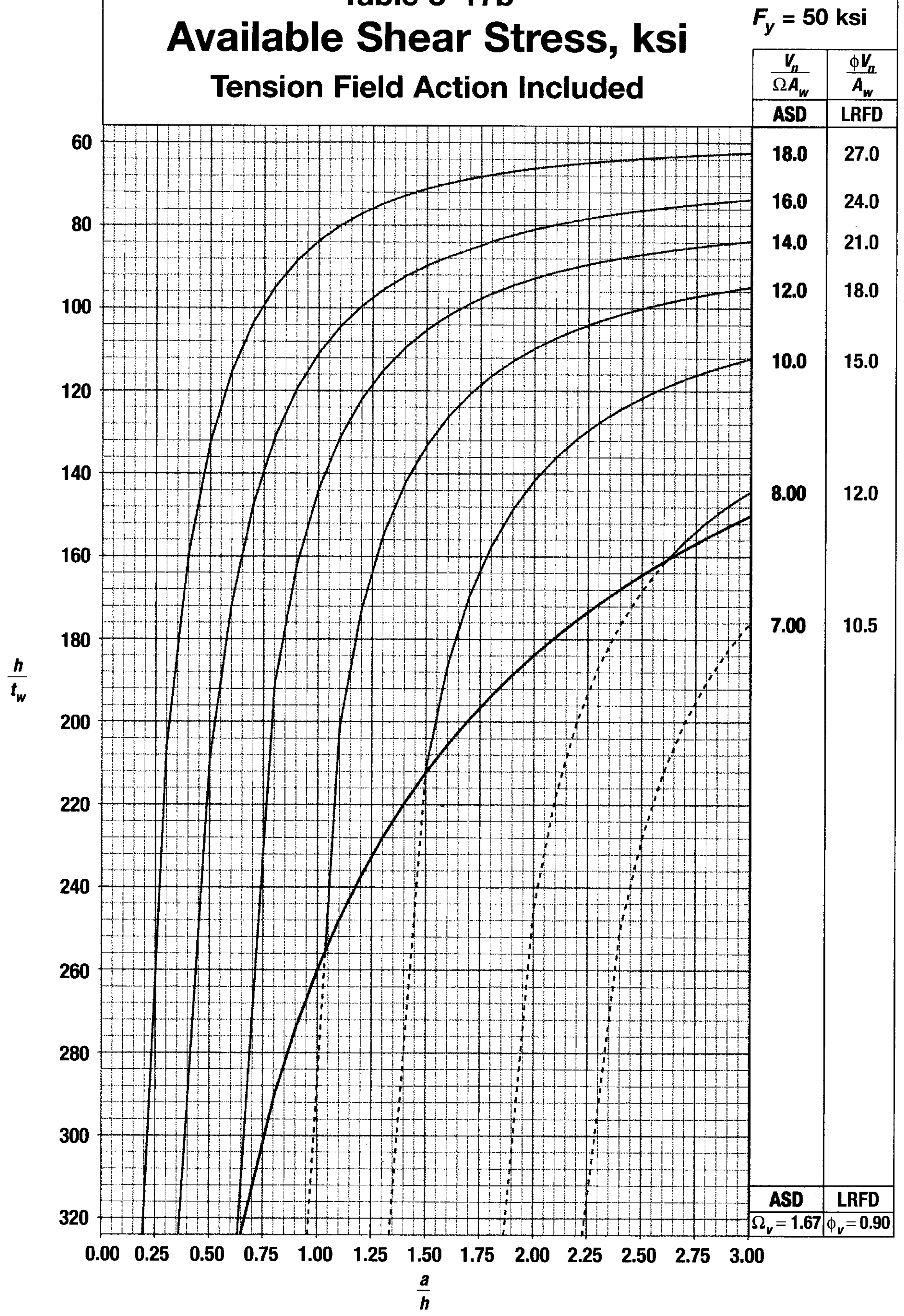


Table 3–17b
Available Shear Stress, ksi
Tension Field Action Included



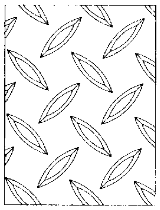


Table 3–18a

Raised Pattern Floor Plate Deflection-Controlled Applications

Recommended Maximum Uniformly Distributed Service Load, lb/ft^2

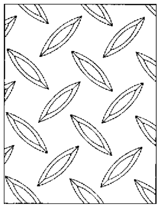
Plate thickness t, in.	Theoretical weight, lb/ft^2	Span, ft					Moment of inertia per ft of width, in.^{4}/ft
		1.5	2	2.5	3	3.5	
1/8	6.15	89.5	37.8	19.3	11.2	7.05	0.00195
3/16	8.70	302	127	65.3	37.8	23.8	0.00659
1/4	11.3	716	302	155	89.5	56.4	0.0156
5/16	13.8	1400	590	302	175	110	0.0305
3/8	16.4	2420	1020	522	302	190	0.0527
1/2	21.5	5730	2420	1240	716	451	0.125
5/8	26.6	11200	4720	2420	1400	881	0.244
3/4	31.7	19300	8160	4180	2420	1520	0.422
7/8	36.8	30700	13000	6630	3840	2420	0.670
1	41.9	45800	19300	9900	5730	3610	1.00
1 1/4	52.1	89500	37800	19300	11200	7050	1.95
1 1/2	62.3	155000	65300	33400	19300	12200	3.38
1 3/4	72.5	246000	104000	53100	30700	19300	5.36
2	82.7	367000	155000	79200	45800	28900	8.00

Plate thickness t, in.	Theoretical weight, lb/ft^2	Span, ft					Moment of inertia per ft of width, in.^{4}/ft
		4	4.5	5	6	7	
3/16	8.70	15.9	11.2	8.16	4.72	2.97	0.00659
1/4	11.3	37.8	26.5	19.3	11.2	7.05	0.0156
5/16	13.8	73.8	51.8	37.8	21.9	13.8	0.0305
3/8	16.4	127	89.5	65.3	37.8	23.8	0.0527
1/2	21.5	302	212	155	89.5	56.4	0.125
5/8	26.6	590	414	302	175	110	0.244
3/4	31.7	1020	716	522	302	190	0.422
7/8	36.8	1620	1140	829	480	302	0.670
1	41.9	2420	1700	1240	716	451	1.00
1 1/4	52.1	4720	3320	2420	1400	881	1.95
1 1/2	62.3	8160	5730	4180	2420	1520	3.38
1 3/4	72.5	13000	9100	6630	3840	2420	5.36
2	82.7	19300	13600	9900	5730	3610	8.00

Note: Material conforms to ASTM A786.

Table 3–18b

Raised Pattern Floor Plate Flexural-Strength-Controlled Applications



Recommended Maximum Uniformly Distributed Load, lb/ft^2

Plate thickness *t*, in.	Theoretical weight, lb/ft^2	Span, ft										Plastic section modulus per ft of width, in.^{3}/ft
		1.5		2		2.5		3		3.5		
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
1/8	6.15	222	333	125	188	79.8	120	55.4	83.3	40.7	61.2	0.0469
3/16	8.70	499	750	281	422	180	270	125	188	91.7	138	0.105
1/4	11.3	887	1330	499	750	319	480	222	333	163	245	0.188
5/16	13.8	1390	2080	780	1170	499	750	347	521	255	383	0.293
3/8	16.4	2000	3000	1120	1690	719	1080	499	750	367	551	0.422
1/2	21.5	3550	5330	2000	3000	1280	1920	887	1330	652	980	0.750
5/8	26.6	5540	8330	3120	4690	2000	3000	1390	2080	1020	1530	1.17
3/4	31.7	7980	12000	4490	6750	2870	4320	2000	3000	1470	2200	1.69
7/8	36.8	10900	16300	6110	9190	3910	5880	2720	4080	2000	3000	2.30
1	41.9	14200	21300	7980	12000	5110	7680	3550	5330	2610	3920	3.00
1 1/4	52.1	22200	33300	12500	18800	7980	12000	5540	8330	4070	6120	4.69
1 1/2	62.3	31900	48000	18000	27000	11500	17300	7980	12000	5870	8820	6.75
1 3/4	72.5	43500	65300	24500	36800	15600	23500	10900	16300	7980	12000	9.19
2	82.7	56800	85300	31900	48000	20400	30700	14200	21300	10400	15700	12.0

Plate thickness *t*, in.	Theoretical weight, lb/ft^2	Span, ft										Plastic section modulus per ft of width, in.^{3}/ft
		4		4.5		5		6		7		
Design		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
3/16	8.70	70.2	105	55.4	83.3	44.9	67.5	31.2	46.9	22.9	34.4	0.105
1/4	11.3	125	188	98.6	148	79.8	120	55.4	83.3	40.7	61.2	0.188
5/16	13.8	195	293	154	231	125	188	86.6	130	63.6	95.7	0.293
3/8	16.4	281	422	222	333	180	270	125	188	91.7	138	0.422
1/2	21.5	499	750	394	593	319	480	222	333	163	245	0.750
5/8	26.6	780	1170	616	926	499	750	347	521	255	383	1.17
3/4	31.7	1120	1690	887	1330	719	1080	499	750	367	551	1.69
7/8	36.8	1530	2300	1210	1810	978	1470	679	1020	499	750	2.30
1	41.9	2000	3000	1580	2370	1280	1920	887	1330	652	980	3.00
1 1/4	52.1	3120	4690	2460	3700	2000	3000	1390	2080	1020	1530	4.69
1 1/2	62.3	4490	6750	3550	5330	2870	4320	2000	3000	1470	2200	6.75
1 3/4	72.5	6110	9190	4830	7260	3910	5880	2720	4080	2000	3000	9.19
2	82.7	7980	12000	6310	9480	5110	7680	3550	5330	2610	3920	12.0

Note: Material conforms to ASTM A786.

Table 3–19
Composite W Shapes
Available Strength in Flexure, kip-ft

W40

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×297	**3320**	4990	TFL	0	4370	**4780**	7180	**4890**	7350	**5000**	7510	**5110**	7680
			2	0.413	3720	**4710**	7080	**4800**	7220	**4890**	7360	**4990**	7490
			3	0.825	3060	**4620**	6950	**4700**	7060	**4780**	7180	**4850**	7290
			4	1.24	2410	**4520**	6800	**4580**	6890	**4640**	6980	**4700**	7070
			BFL	1.65	1760	**4410**	6630	**4460**	6700	**4500**	6760	**4540**	6830
			6	4.59	1430	**4330**	6510	**4370**	6570	**4400**	6620	**4440**	6670
			7	8.17	1090	**4190**	6300	**4220**	6340	**4250**	6380	**4270**	6420
W40×294	**3170**	4760	TFL	0	4310	**4780**	7180	**4890**	7340	**4990**	7500	**5100**	7670
			2	0.483	3730	**4710**	7080	**4810**	7220	**4900**	7360	**4990**	7500
			3	0.965	3150	**4630**	6970	**4710**	7080	**4790**	7200	**4870**	7320
			4	1.45	2580	**4540**	6830	**4610**	6920	**4670**	7020	**4730**	7120
			BFL	1.93	2000	**4440**	6670	**4480**	6740	**4530**	6820	**4580**	6890
			6	5.69	1540	**4310**	6480	**4350**	6530	**4390**	6590	**4420**	6650
			7	10.00	1080	**4080**	6140	**4110**	6180	**4140**	6220	**4170**	6260
W40×278	**2970**	4460	TFL	0	4100	**4520**	6790	**4620**	6940	**4720**	7100	**4820**	7250
			2	0.453	3560	**4460**	6700	**4540**	6830	**4630**	6960	**4720**	7100
			3	0.905	3020	**4380**	6590	**4460**	6700	**4530**	6820	**4610**	6930
			4	1.36	2470	**4300**	6460	**4360**	6550	**4420**	6650	**4480**	6740
			BFL	1.81	1930	**4200**	6320	**4250**	6390	**4300**	6460	**4350**	6530
			6	5.65	1480	**4080**	6130	**4120**	6190	**4150**	6240	**4190**	6300
			7	10.1	1020	**3850**	5790	**3880**	5830	**3910**	5870	**3930**	5910
W40×277	**3120**	4690	TFL	0	4070	**4440**	6670	**4540**	6820	**4640**	6970	**4740**	7130
			2	0.394	3450	**4370**	6560	**4450**	6690	**4540**	6820	**4630**	6950
			3	0.788	2820	**4290**	6440	**4360**	6550	**4430**	6660	**4500**	6760
			4	1.18	2200	**4190**	6300	**4250**	6390	**4300**	6470	**4360**	6550
			BFL	1.58	1580	**4090**	6150	**4130**	6210	**4170**	6260	**4210**	6320
			6	4.22	1300	**4030**	6050	**4060**	6100	**4090**	6150	**4120**	6200
			7	7.59	1020	**3920**	5890	**3940**	5920	**3970**	5960	**3990**	6000
W40×264	**2820**	4240	TFL	0	3880	**4260**	6400	**4350**	6540	**4450**	6690	**4550**	6830
			2	0.433	3360	**4200**	6310	**4280**	6440	**4370**	6560	**4450**	6690
			3	0.865	2850	**4130**	6210	**4200**	6320	**4270**	6420	**4340**	6530
			4	1.30	2330	**4050**	6090	**4110**	6180	**4170**	6270	**4230**	6350
			BFL	1.73	1810	**3960**	5950	**4010**	6020	**4050**	6090	**4100**	6160
			6	5.50	1390	**3850**	5790	**3880**	5840	**3920**	5890	**3950**	5940
			7	9.90	969	**3650**	5480	**3670**	5520	**3690**	5550	**3720**	5590

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W40

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×297	5220	7840	5330	8000	5430	8170	5540	8330	5650	8500	5760	8660	5870	8820
	5080	7630	5170	7770	5260	7910	5360	8050	5450	8190	5540	8330	5640	8470
	4930	7410	5010	7520	5080	7640	5160	7750	5230	7870	5310	7980	5390	8100
	4760	7160	4820	7250	4890	7340	4950	7430	5010	7520	5070	7610	5130	7700
	4590	6900	4630	6960	4680	7030	4720	7090	4760	7160	4810	7230	4850	7290
	4480	6730	4510	6780	4550	6830	4580	6890	4620	6940	4650	6990	4690	7050
	4300	6470	4330	6510	4360	6550	4380	6590	4410	6630	4440	6670	4460	6710
W40×294	5210	7830	5320	7990	5420	8150	5530	8310	5640	8480	5750	8640	5850	8800
	5090	7640	5180	7780	5270	7920	5370	8060	5460	8200	5550	8340	5640	8480
	4950	7440	5030	7560	5110	7670	5180	7790	5260	7910	5340	8030	5420	8150
	4800	7210	4860	7310	4930	7410	4990	7500	5060	7600	5120	7700	5180	7790
	4630	6970	4680	7040	4730	7120	4780	7190	4830	7270	4880	7340	4930	7410
	4460	6710	4500	6760	4540	6820	4580	6880	4620	6940	4650	7000	4690	7050
	4190	6300	4220	6340	4250	6380	4270	6420	4300	6460	4330	6500	4350	6540
W40×278	4930	7400	5030	7560	5130	7710	5230	7860	5330	8020	5440	8170	5540	8330
	4810	7230	4900	7360	4990	7500	5080	7630	5170	7770	5260	7900	5340	8030
	4680	7040	4760	7150	4840	7270	4910	7380	4990	7490	5060	7610	5140	7720
	4550	6830	4610	6930	4670	7020	4730	7110	4790	7200	4850	7300	4920	7390
	4390	6610	4440	6680	4490	6750	4540	6820	4590	6900	4640	6970	4680	7040
	4230	6350	4260	6410	4300	6460	4340	6520	4370	6580	4410	6630	4450	6690
	3960	5950	3980	5980	4010	6020	4030	6060	4060	6100	4080	6140	4110	6180
W40×277	4840	7280	4940	7430	5050	7580	5150	7740	5250	7890	5350	8040	5450	8190
	4710	7080	4800	7210	4880	7340	4970	7470	5060	7600	5140	7730	5230	7860
	4570	6870	4640	6970	4710	7080	4780	7180	4850	7290	4920	7400	4990	7500
	4410	6630	4470	6720	4520	6800	4580	6880	4630	6960	4690	7050	4740	7130
	4250	6380	4290	6440	4330	6500	4370	6560	4400	6620	4440	6680	4480	6740
	4160	6250	4190	6290	4220	6340	4250	6390	4290	6440	4320	6490	4350	6540
	4020	6040	4040	6080	4070	6110	4090	6150	4120	6190	4140	6230	4170	6270
W40×264	4640	6980	4740	7130	4840	7270	4930	7420	5030	7560	5130	7710	5220	7850
	4540	6820	4620	6940	4700	7070	4790	7190	4870	7320	4950	7450	5040	7570
	4420	6640	4490	6740	4560	6850	4630	6960	4700	7060	4770	7170	4840	7280
	4280	6440	4340	6530	4400	6610	4460	6700	4520	6790	4580	6880	4630	6960
	4140	6230	4190	6290	4230	6360	4280	6430	4320	6500	4370	6570	4410	6630
	3990	6000	4020	6050	4060	6100	4090	6150	4130	6200	4160	6260	4200	6310
	3740	5630	3770	5660	3790	5700	3820	5730	3840	5770	3860	5810	3890	5840

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

W40

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×249	**2790**	4200	TFL	0	3670	**3970**	5960	**4060**	6100	**4150**	6240	**4240**	6380
			2	0.355	3110	**3910**	5870	**3990**	5990	**4060**	6110	**4140**	6220
			3	0.710	2550	**3840**	5770	**3900**	5860	**3960**	5960	**4030**	6050
			4	1.07	1990	**3760**	5650	**3810**	5720	**3860**	5800	**3910**	5870
			BFL	1.42	1430	**3670**	5510	**3700**	5560	**3740**	5620	**3770**	5670
			6	4.04	1170	**3610**	5430	**3640**	5470	**3670**	5510	**3700**	5560
			7	7.47	917	**3510**	5280	**3530**	5310	**3560**	5350	**3580**	5380
W40×235	**2520**	3790	TFL	0	3450	**3760**	5650	**3850**	5780	**3930**	5910	**4020**	6040
			2	0.394	2980	**3710**	5570	**3780**	5690	**3860**	5800	**3930**	5910
			3	0.788	2510	**3650**	5480	**3710**	5580	**3770**	5670	**3840**	5770
			4	1.18	2040	**3580**	5380	**3630**	5450	**3680**	5530	**3730**	5610
			BFL	1.58	1580	**3500**	5260	**3540**	5320	**3580**	5380	**3620**	5440
			6	5.16	1220	**3410**	5120	**3440**	5170	**3470**	5220	**3500**	5260
			7	9.46	862	**3240**	4880	**3270**	4910	**3290**	4940	**3310**	4970
W40×215	**2410**	3620	TFL	0	3170	**3400**	5110	**3480**	5230	**3560**	5350	**3640**	5460
			2	0.305	2690	**3350**	5030	**3410**	5130	**3480**	5230	**3550**	5330
			3	0.610	2210	**3290**	4940	**3340**	5020	**3400**	5110	**3450**	5190
			4	0.915	1730	**3220**	4840	**3260**	4910	**3310**	4970	**3350**	5040
			BFL	1.22	1250	**3150**	4730	**3180**	4780	**3210**	4830	**3240**	4870
			6	3.80	1020	**3100**	4660	**3130**	4700	**3150**	4740	**3180**	4780
			7	7.30	792	**3020**	4530	**3040**	4560	**3050**	4590	**3070**	4620
W40×211	**2260**	3400	TFL	0	3100	**3360**	5040	**3430**	5160	**3510**	5280	**3590**	5390
			2	0.354	2680	**3310**	4980	**3380**	5080	**3440**	5180	**3510**	5280
			3	0.708	2270	**3260**	4900	**3310**	4980	**3370**	5070	**3430**	5150
			4	1.06	1850	**3200**	4810	**3240**	4880	**3290**	4940	**3340**	5010
			BFL	1.42	1430	**3130**	4700	**3170**	4760	**3200**	4810	**3240**	4870
			6	4.98	1100	**3050**	4590	**3080**	4630	**3110**	4670	**3130**	4710
			7	9.35	775	**2900**	4360	**2920**	4390	**2940**	4420	**2960**	4450
W40×199	**2170**	3260	TFL	0	2920	**3110**	4680	**3190**	4790	**3260**	4900	**3330**	5010
			2	0.266	2510	**3070**	4610	**3130**	4710	**3190**	4800	**3260**	4890
			3	0.533	2090	**3020**	4540	**3070**	4620	**3120**	4690	**3180**	4770
			4	0.799	1670	**2960**	4450	**3000**	4520	**3050**	4580	**3090**	4640
			BFL	1.07	1250	**2900**	4360	**2930**	4410	**2960**	4460	**3000**	4500
			6	4.11	989	**2850**	4280	**2870**	4320	**2900**	4350	**2920**	4390
			7	8.09	731	**2740**	4120	**2760**	4150	**2780**	4180	**2800**	4210

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W40

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×249	**4330**	6510	**4430**	6650	**4520**	6790	**4610**	6930	**4700**	7060	**4790**	7200	**4880**	7340
	4220	6340	**4300**	6460	**4370**	6570	**4450**	6690	**4530**	6810	**4610**	6920	**4680**	7040
	4090	6150	**4160**	6250	**4220**	6340	**4280**	6440	**4350**	6530	**4410**	6630	**4470**	6720
	3960	5940	**4000**	6020	**4050**	6090	**4100**	6170	**4150**	6240	**4200**	6320	**4250**	6390
	3810	5720	**3840**	5780	**3880**	5830	**3920**	5890	**3950**	5940	**3990**	5990	**4020**	6050
	3730	5600	**3760**	5650	**3790**	5690	**3810**	5730	**3840**	5780	**3870**	5820	**3900**	5870
	3600	5410	**3620**	5450	**3650**	5480	**3670**	5520	**3690**	5550	**3720**	5590	**3740**	5620
W40×235	**4100**	6170	**4190**	6300	**4280**	6430	**4360**	6560	**4450**	6690	**4530**	6810	**4620**	6940
	4010	6020	**4080**	6130	**4150**	6240	**4230**	6360	**4300**	6470	**4380**	6580	**4450**	6690
	3900	5860	**3960**	5950	**4020**	6050	**4090**	6140	**4150**	6240	**4210**	6330	**4270**	6420
	3780	5680	**3830**	5760	**3880**	5840	**3930**	5910	**3990**	5990	**4040**	6070	**4090**	6140
	3660	5500	**3700**	5560	**3740**	5610	**3770**	5670	**3810**	5730	**3850**	5790	**3890**	5850
	3530	5310	**3560**	5350	**3590**	5400	**3620**	5440	**3650**	5490	**3680**	5540	**3710**	5580
	3330	5000	**3350**	5040	**3370**	5070	**3390**	5100	**3420**	5130	**3440**	5170	**3460**	5200
W40×215	**3710**	5580	**3790**	5700	**3870**	5820	**3950**	5940	**4030**	6060	**4110**	6180	**4190**	6300
	3610	5430	**3680**	5530	**3750**	5630	**3820**	5740	**3880**	5840	**3950**	5940	**4020**	6040
	3510	5270	**3560**	5360	**3620**	5440	**3670**	5520	**3730**	5600	**3780**	5690	**3840**	5770
	3390	5100	**3440**	5170	**3480**	5230	**3520**	5300	**3570**	5360	**3610**	5430	**3650**	5490
	3270	4920	**3300**	4970	**3330**	5010	**3370**	5060	**3400**	5110	**3430**	5150	**3460**	5200
	3200	4810	**3230**	4850	**3250**	4890	**3280**	4930	**3300**	4970	**3330**	5010	**3360**	5040
	3090	4650	**3110**	4680	**3130**	4710	**3150**	4740	**3170**	4770	**3190**	4800	**3210**	4830
W40×211	**3670**	5510	**3740**	5630	**3820**	5740	**3900**	5860	**3970**	5970	**4050**	6090	**4130**	6210
	3580	5380	**3650**	5480	**3710**	5580	**3780**	5680	**3850**	5780	**3910**	5880	**3980**	5980
	3480	5240	**3540**	5320	**3600**	5410	**3650**	5490	**3710**	5580	**3770**	5660	**3820**	5750
	3380	5080	**3430**	5150	**3470**	5220	**3520**	5290	**3570**	5360	**3610**	5430	**3660**	5500
	3270	4920	**3310**	4970	**3340**	5030	**3380**	5080	**3420**	5130	**3450**	5190	**3490**	5240
	3160	4750	**3190**	4790	**3220**	4830	**3240**	4870	**3270**	4920	**3300**	4960	**3330**	5000
	2980	4480	**3000**	4510	**3020**	4530	**3040**	4560	**3060**	4590	**3080**	4620	**3090**	4650
W40×199	**3410**	5120	**3480**	5230	**3550**	5340	**3620**	5450	**3700**	5560	**3770**	5670	**3840**	5780
	3320	4990	**3380**	5080	**3440**	5180	**3510**	5270	**3570**	5360	**3630**	5460	**3690**	5550
	3230	4850	**3280**	4930	**3330**	5010	**3380**	5090	**3440**	5160	**3490**	5240	**3540**	5320
	3130	4700	**3170**	4770	**3210**	4830	**3250**	4890	**3300**	4950	**3340**	5020	**3380**	5080
	3030	4550	**3060**	4600	**3090**	4640	**3120**	4690	**3150**	4740	**3180**	4780	**3210**	4830
	2950	4430	**2970**	4470	**3000**	4500	**3020**	4540	**3050**	4580	**3070**	4610	**3090**	4650
	2820	4230	**2830**	4260	**2850**	4290	**2870**	4310	**2890**	4340	**2910**	4370	**2930**	4400

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3-3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W40–W36

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×183	**1930**	2900	TFL	0	2670	**2860**	4300	**2930**	4400	**2990**	4500	**3060**	4600
			2	0.300	2310	**2820**	4240	**2880**	4330	**2940**	4410	**2990**	4500
			3	0.600	1960	**2780**	4170	**2830**	4250	**2880**	4320	**2920**	4390
			4	0.900	1600	**2730**	4100	**2770**	4160	**2810**	4220	**2850**	4280
			BFL	1.20	1250	**2670**	4020	**2710**	4070	**2740**	4110	**2770**	4160
			6	4.76	958	**2610**	3920	**2630**	3960	**2660**	3990	**2680**	4030
			7	9.24	666	**2480**	3720	**2490**	3750	**2510**	3770	**2530**	3800
W40×167	**1730**	2600	TFL	0	2460	**2610**	3930	**2670**	4020	**2730**	4110	**2800**	4200
			2	0.256	2160	**2580**	3880	**2630**	3960	**2690**	4040	**2740**	4120
			3	0.513	1850	**2540**	3820	**2590**	3890	**2640**	3960	**2680**	4030
			4	0.769	1550	**2500**	3760	**2540**	3820	**2580**	3880	**2620**	3940
			BFL	1.03	1250	**2460**	3700	**2490**	3740	**2520**	3790	**2550**	3840
			6	4.97	931	**2390**	3590	**2410**	3620	**2430**	3660	**2460**	3690
			7	9.84	614	**2240**	3370	**2250**	3390	**2270**	3410	**2280**	3430
W40×149	**1490**	2240	TFL	0	2190	**2310**	3470	**2360**	3550	**2420**	3630	**2470**	3710
			2	0.208	1950	**2280**	3430	**2330**	3500	**2380**	3570	**2430**	3650
			3	0.415	1700	**2250**	3390	**2300**	3450	**2340**	3510	**2380**	3580
			4	0.623	1460	**2220**	3340	**2260**	3390	**2290**	3450	**2330**	3500
			BFL	0.830	1210	**2190**	3290	**2220**	3340	**2250**	3380	**2280**	3430
			6	5.14	879	**2110**	3180	**2140**	3210	**2160**	3240	**2180**	3280
			7	10.4	548	**1950**	2930	**1970**	2950	**1980**	2970	**1990**	3000
W36×302	**3190**	4800	TFL	0	4440	**4580**	6880	**4690**	7050	**4800**	7220	**4910**	7380
			2	0.420	3740	**4500**	6770	**4600**	6910	**4690**	7050	**4780**	7190
			3	0.840	3040	**4410**	6630	**4490**	6740	**4560**	6860	**4640**	6970
			4	1.26	2340	**4300**	6470	**4360**	6560	**4420**	6650	**4480**	6730
			BFL	1.68	1640	**4180**	6290	**4220**	6350	**4270**	6410	**4310**	6470
			6	4.09	1380	**4120**	6200	**4160**	6250	**4190**	6300	**4230**	6350
			7	6.91	1110	**4020**	6050	**4050**	6090	**4080**	6130	**4110**	6170
W36×282	**2970**	4460	TFL	0	4150	**4250**	6390	**4360**	6550	**4460**	6700	**4560**	6860
			2	0.393	3500	**4180**	6290	**4270**	6420	**4360**	6550	**4440**	6680
			3	0.785	2840	**4100**	6160	**4170**	6270	**4240**	6370	**4310**	6480
			4	1.18	2190	**4000**	6010	**4060**	6100	**4110**	6180	**4170**	6260
			BFL	1.57	1540	**3890**	5850	**3930**	5910	**3970**	5960	**4010**	6020
			6	3.99	1290	**3830**	5760	**3870**	5810	**3900**	5860	**3930**	5910
			7	6.84	1040	**3740**	5620	**3770**	5660	**3790**	5700	**3820**	5740

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite WShapes
Available Strength in Flexure, kip-ft

W40–W36

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W40×183	**3120**	4700	**3190**	4800	**3260**	4900	**3320**	5000	**3390**	5100	**3460**	5200	**3520**	5300
	3050	4590	**3110**	4670	**3170**	4760	**3220**	4850	**3280**	4930	**3340**	5020	**3400**	5110
	2970	4470	**3020**	4540	**3070**	4620	**3120**	4690	**3170**	4760	**3220**	4840	**3270**	4910
	2890	4340	**2930**	4400	**2970**	4460	**3010**	4520	**3050**	4580	**3090**	4640	**3130**	4700
	2800	4210	**2830**	4250	**2860**	4300	**2890**	4350	**2920**	4400	**2960**	4440	**2990**	4490
	2700	4060	**2730**	4100	**2750**	4140	**2780**	4170	**2800**	4210	**2820**	4240	**2850**	4280
	2540	3820	**2560**	3850	**2580**	3870	**2590**	3900	**2610**	3920	**2630**	3950	**2640**	3970
W40×167	**2860**	4290	**2920**	4390	**2980**	4480	**3040**	4570	**3100**	4660	**3160**	4760	**3230**	4850
	2790	4200	**2850**	4280	**2900**	4360	**2960**	4440	**3010**	4520	**3060**	4600	**3120**	4690
	2730	4100	**2770**	4170	**2820**	4240	**2870**	4310	**2910**	4380	**2960**	4450	**3010**	4520
	2660	4000	**2700**	4050	**2740**	4110	**2770**	4170	**2810**	4230	**2850**	4290	**2890**	4340
	2580	3880	**2620**	3930	**2650**	3980	**2680**	4030	**2710**	4070	**2740**	4120	**2770**	4170
	2480	3730	**2500**	3760	**2530**	3800	**2550**	3830	**2570**	3870	**2600**	3900	**2620**	3940
	2300	3460	**2320**	3480	**2330**	3500	**2350**	3530	**2360**	3550	**2380**	3570	**2390**	3600
W40×149	**2530**	3800	**2580**	3880	**2640**	3960	**2690**	4040	**2740**	4120	**2800**	4210	**2850**	4290
	2480	3720	**2520**	3790	**2570**	3870	**2620**	3940	**2670**	4010	**2720**	4090	**2770**	4160
	2420	3640	**2470**	3710	**2510**	3770	**2550**	3830	**2590**	3900	**2630**	3960	**2680**	4020
	2370	3560	**2400**	3610	**2440**	3670	**2480**	3720	**2510**	3780	**2550**	3830	**2590**	3890
	2310	3470	**2340**	3520	**2370**	3560	**2400**	3610	**2430**	3650	**2460**	3700	**2490**	3740
	2200	3310	**2220**	3340	**2240**	3370	**2270**	3410	**2290**	3440	**2310**	3470	**2330**	3510
	2010	3020	**2020**	3040	**2030**	3060	**2050**	3080	**2060**	3100	**2070**	3120	**2090**	3140
W36×302	**5020**	7550	**5130**	7720	**5250**	7880	**5360**	8050	**5470**	8220	**5580**	8380	**5690**	8550
	4880	7330	**4970**	7470	**5060**	7610	**5160**	7750	**5250**	7890	**5340**	8030	**5440**	8170
	4710	7090	**4790**	7200	**4870**	7310	**4940**	7430	**5020**	7540	**5090**	7660	**5170**	7770
	4540	6820	**4600**	6910	**4660**	7000	**4710**	7080	**4770**	7170	**4830**	7260	**4890**	7350
	4350	6530	**4390**	6600	**4430**	6660	**4470**	6720	**4510**	6780	**4550**	6840	**4590**	6900
	4260	6400	**4290**	6450	**4330**	6510	**4360**	6560	**4400**	6610	**4430**	6660	**4470**	6710
	4130	6210	**4160**	6250	**4190**	6300	**4220**	6340	**4240**	6380	**4270**	6420	**4300**	6460
W36×282	**4670**	7020	**4770**	7170	**4870**	7330	**4980**	7480	**5080**	7640	**5190**	7790	**5290**	7950
	4530	6810	**4620**	6940	**4710**	7070	**4790**	7200	**4880**	7340	**4970**	7470	**5050**	7600
	4380	6590	**4450**	6690	**4520**	6800	**4590**	6910	**4670**	7010	**4740**	7120	**4810**	7230
	4220	6340	**4270**	6420	**4330**	6510	**4380**	6590	**4440**	6670	**4490**	6750	**4550**	6840
	4050	6080	**4080**	6140	**4120**	6200	**4160**	6250	**4200**	6310	**4240**	6370	**4280**	6430
	3960	5960	**4000**	6010	**4030**	6050	**4060**	6100	**4090**	6150	**4120**	6200	**4160**	6250
	3850	5780	**3870**	5820	**3900**	5860	**3920**	5900	**3950**	5940	**3970**	5970	**4000**	6010

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W36

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	Y1[a]	ΣQ_n	Y2[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×262	2740	4130	TFL	0	3850	3920	5900	4020	6040	4110	6180	4210	6330
			2	0.360	3250	3860	5800	3940	5920	4020	6040	4100	6160
			3	0.720	2660	3780	5680	3850	5780	3910	5880	3980	5980
			4	1.08	2060	3700	5550	3750	5630	3800	5710	3850	5790
			BFL	1.44	1470	3600	5410	3640	5460	3670	5520	3710	5570
			6	3.98	1210	3540	5320	3570	5370	3600	5420	3630	5460
			7	6.97	962	3450	5180	3470	5220	3500	5260	3520	5290
W36×256	2590	3900	TFL	0	3770	3890	5850	3990	6000	4080	6140	4180	6280
			2	0.433	3240	3840	5770	3920	5890	4000	6010	4080	6130
			3	0.865	2710	3770	5660	3830	5760	3900	5860	3970	5970
			4	1.30	2180	3690	5540	3740	5620	3790	5700	3850	5780
			BFL	1.73	1650	3590	5400	3630	5460	3680	5520	3720	5590
			6	5.19	1300	3500	5260	3530	5310	3560	5360	3600	5400
			7	8.90	942	3340	5020	3360	5050	3380	5090	3410	5120
W36×247	2570	3860	TFL	0	3630	3680	5530	3770	5670	3860	5800	3950	5940
			2	0.338	3070	3620	5440	3700	5560	3770	5670	3850	5790
			3	0.675	2510	3550	5340	3610	5430	3670	5520	3740	5620
			4	1.01	1950	3470	5220	3520	5290	3570	5360	3620	5440
			BFL	1.35	1400	3380	5080	3420	5140	3450	5190	3490	5240
			6	3.93	1150	3330	5000	3360	5050	3390	5090	3410	5130
			7	7.00	907	3240	4870	3260	4900	3280	4930	3300	4970
W36×232	2340	3510	TFL	0	3410	3490	5250	3580	5380	3660	5510	3750	5630
			2	0.393	2930	3440	5170	3510	5280	3590	5390	3660	5500
			3	0.785	2450	3380	5080	3440	5170	3500	5260	3560	5360
			4	1.18	1980	3310	4970	3360	5050	3410	5120	3460	5200
			BFL	1.57	1500	3230	4850	3270	4910	3300	4970	3340	5020
			6	5.03	1180	3150	4730	3180	4770	3200	4820	3230	4860
			7	8.77	851	3000	4510	3020	4540	3040	4570	3060	4610
W36×231	2400	3610	TFL	0	3400	3440	5170	3520	5300	3610	5430	3690	5550
			2	0.315	2890	3380	5090	3460	5190	3530	5300	3600	5410
			3	0.630	2370	3320	4990	3380	5080	3440	5170	3500	5260
			4	0.945	1850	3250	4880	3290	4950	3340	5020	3390	5090
			BFL	1.26	1330	3170	4760	3200	4810	3230	4860	3270	4910
			6	3.90	1090	3120	4680	3140	4720	3170	4760	3200	4810
			7	7.05	851	3030	4550	3050	4580	3070	4610	3090	4640

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W36

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×262	4310	6470	4400	6620	4500	6760	4590	6910	4690	7050	4790	7190	4880	7340
	4180	6290	4260	6410	4340	6530	4430	6650	4510	6770	4590	6900	4670	7020
	4050	6080	4110	6180	4180	6280	4250	6380	4310	6480	4380	6580	4450	6680
	3900	5860	3950	5940	4000	6020	4060	6100	4110	6170	4160	6250	4210	6330
	3750	5630	3780	5680	3820	5740	3860	5800	3890	5850	3930	5910	3970	5960
	3660	5510	3690	5550	3720	5600	3750	5640	3780	5690	3810	5730	3840	5780
	3550	5330	3570	5360	3590	5400	3620	5440	3640	5470	3670	5510	3690	5540
W36×256	4270	6420	4370	6560	4460	6700	4550	6840	4650	6980	4740	7130	4840	7270
	4160	6250	4240	6370	4320	6500	4400	6620	4480	6740	4560	6860	4640	6980
	4040	6070	4100	6170	4170	6270	4240	6370	4310	6470	4380	6580	4440	6680
	3900	5870	3960	5950	4010	6030	4070	6110	4120	6190	4180	6280	4230	6360
	3760	5650	3800	5710	3840	5770	3880	5830	3920	5900	3960	5960	4000	6020
	3630	5450	3660	5500	3690	5550	3730	5600	3760	5650	3790	5700	3820	5750
	3430	5160	3460	5190	3480	5230	3500	5260	3530	5300	3550	5330	3570	5370
W36×247	4040	6070	4130	6210	4220	6350	4310	6480	4400	6620	4490	6750	4580	6890
	3930	5900	4000	6020	4080	6130	4160	6250	4230	6360	4310	6480	4390	6590
	3800	5710	3860	5810	3930	5900	3990	5990	4050	6090	4110	6180	4180	6280
	3670	5510	3710	5580	3760	5660	3810	5730	3860	5800	3910	5880	3960	5950
	3520	5290	3560	5350	3590	5400	3630	5450	3660	5500	3700	5560	3730	5610
	3440	5180	3470	5220	3500	5260	3530	5300	3560	5350	3590	5390	3620	5430
	3330	5000	3350	5040	3370	5070	3400	5100	3420	5140	3440	5170	3460	5210
W36×232	3830	5760	3920	5890	4000	6020	4090	6140	4170	6270	4260	6400	4340	6530
	3730	5610	3810	5720	3880	5830	3950	5940	4030	6050	4100	6160	4170	6270
	3620	5450	3690	5540	3750	5630	3810	5720	3870	5820	3930	5910	3990	6000
	3510	5270	3560	5340	3610	5420	3650	5490	3700	5570	3750	5640	3800	5720
	3380	5080	3420	5130	3450	5190	3490	5250	3530	5300	3570	5360	3600	5420
	3260	4910	3290	4950	3320	4990	3350	5040	3380	5080	3410	5130	3440	5170
	3090	4640	3110	4670	3130	4700	3150	4730	3170	4770	3190	4800	3210	4830
W36×231	3780	5680	3860	5810	3950	5940	4030	6060	4120	6190	4200	6320	4290	6450
	3670	5520	3740	5630	3820	5740	3890	5840	3960	5950	4030	6060	4100	6170
	3560	5340	3620	5430	3670	5520	3730	5610	3790	5700	3850	5790	3910	5880
	3430	5160	3480	5230	3520	5300	3570	5370	3620	5440	3660	5510	3710	5570
	3300	4960	3330	5010	3370	5060	3400	5110	3430	5160	3470	5210	3500	5260
	3220	4850	3250	4890	3280	4930	3310	4970	3330	5010	3360	5050	3390	5090
	3110	4680	3130	4710	3150	4740	3170	4770	3200	4800	3220	4840	3240	4870

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W36

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×210	**2080**	3120	TFL	0	3090	**3140**	4720	**3210**	4830	**3290**	4950	**3370**	5060
			2	0.340	2680	**3090**	4650	**3160**	4750	**3230**	4850	**3290**	4950
			3	0.680	2260	**3040**	4570	**3100**	4660	**3150**	4740	**3210**	4820
			4	1.02	1850	**2980**	4480	**3030**	4550	**3070**	4620	**3120**	4690
			BFL	1.36	1430	**2920**	4380	**2950**	4440	**2990**	4490	**3020**	4540
			6	5.05	1100	**2830**	4260	**2860**	4300	**2890**	4340	**2920**	4380
			7	9.04	773	**2680**	4030	**2700**	4060	**2720**	4090	**2740**	4120
W36×194	**1910**	2880	TFL	0	2850	**2880**	4330	**2950**	4430	**3020**	4540	**3090**	4650
			2	0.315	2470	**2840**	4270	**2900**	4360	**2960**	4450	**3020**	4540
			3	0.630	2090	**2790**	4200	**2840**	4270	**2900**	4350	**2950**	4430
			4	0.945	1710	**2740**	4120	**2780**	4180	**2820**	4240	**2870**	4310
			BFL	1.26	1320	**2680**	4030	**2710**	4080	**2740**	4130	**2780**	4170
			6	4.94	1020	**2600**	3910	**2630**	3950	**2650**	3990	**2680**	4030
			7	8.93	713	**2470**	3710	**2490**	3740	**2500**	3760	**2520**	3790
W36×182	**1790**	2690	TFL	0	2680	**2700**	4050	**2760**	4150	**2830**	4260	**2900**	4360
			2	0.295	2320	**2660**	4000	**2720**	4080	**2780**	4170	**2830**	4260
			3	0.590	1970	**2620**	3930	**2660**	4010	**2710**	4080	**2760**	4150
			4	0.885	1610	**2570**	3860	**2610**	3920	**2650**	3980	**2690**	4040
			BFL	1.18	1260	**2510**	3780	**2540**	3820	**2580**	3870	**2610**	3920
			6	4.88	963	**2440**	3670	**2470**	3710	**2490**	3740	**2510**	3780
			7	8.92	670	**2310**	3480	**2330**	3500	**2350**	3530	**2360**	3550
W36×170	**1670**	2510	TFL	0	2500	**2510**	3770	**2570**	3860	**2630**	3960	**2700**	4050
			2	0.275	2170	**2470**	3720	**2530**	3800	**2580**	3880	**2640**	3960
			3	0.550	1840	**2430**	3660	**2480**	3730	**2520**	3790	**2570**	3860
			4	0.825	1510	**2390**	3590	**2430**	3650	**2460**	3700	**2500**	3760
			BFL	1.10	1180	**2340**	3520	**2370**	3560	**2400**	3610	**2430**	3650
			6	4.81	902	**2270**	3420	**2300**	3450	**2320**	3490	**2340**	3520
			7	8.88	626	**2150**	3230	**2170**	3260	**2180**	3280	**2200**	3300
W36×160	**1560**	2340	TFL	0	2350	**2350**	3530	**2410**	3620	**2470**	3710	**2520**	3790
			2	0.255	2050	**2320**	3480	**2370**	3560	**2420**	3630	**2470**	3710
			3	0.510	1740	**2280**	3430	**2320**	3490	**2370**	3560	**2410**	3620
			4	0.765	1430	**2240**	3370	**2270**	3420	**2310**	3470	**2350**	3530
			BFL	1.02	1130	**2190**	3300	**2220**	3340	**2250**	3380	**2280**	3430
			6	4.80	858	**2130**	3200	**2150**	3240	**2170**	3270	**2200**	3300
			7	8.96	588	**2010**	3020	**2030**	3050	**2040**	3070	**2060**	3090

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W36

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×210	3450	5180	3520	5300	3600	5410	3680	5530	3750	5640	3830	5760	3910	5870
	3360	5050	3430	5150	3490	5250	3560	5350	3630	5450	3690	5550	3760	5650
	3270	4910	3320	4990	3380	5080	3440	5160	3490	5250	3550	5330	3610	5420
	3170	4760	3210	4830	3260	4900	3300	4970	3350	5040	3400	5110	3440	5180
	3060	4600	3090	4650	3130	4700	3170	4760	3200	4810	3240	4870	3270	4920
	2940	4420	2970	4460	3000	4510	3030	4550	3050	4590	3080	4630	3110	4670
	2760	4150	2780	4180	2800	4210	2820	4240	2840	4270	2860	4290	2880	4320
W36×194	3160	4760	3230	4860	3310	4970	3380	5080	3450	5180	3520	5290	3590	5400
	3080	4640	3150	4730	3210	4820	3270	4910	3330	5010	3390	5100	3450	5190
	3000	4510	3050	4590	3100	4660	3160	4740	3210	4820	3260	4900	3310	4980
	2910	4370	2950	4440	2990	4500	3040	4560	3080	4630	3120	4690	3160	4760
	2810	4220	2840	4270	2880	4320	2910	4370	2940	4420	2980	4470	3010	4520
	2710	4070	2730	4100	2760	4140	2780	4180	2810	4220	2830	4260	2860	4300
	2540	3820	2560	3840	2570	3870	2590	3900	2610	3920	2630	3950	2650	3980
W36×182	2960	4460	3030	4560	3100	4660	3170	4760	3230	4860	3300	4960	3370	5060
	2890	4350	2950	4430	3010	4520	3060	4610	3120	4690	3180	4780	3240	4870
	2810	4230	2860	4300	2910	4370	2960	4450	3010	4520	3060	4600	3110	4670
	2730	4100	2770	4160	2810	4220	2850	4280	2890	4340	2930	4400	2970	4460
	2640	3970	2670	4010	2700	4060	2730	4110	2760	4150	2800	4200	2830	4250
	2540	3820	2560	3850	2590	3890	2610	3920	2630	3960	2660	4000	2680	4030
	2380	3580	2400	3600	2410	3630	2430	3650	2450	3680	2460	3700	2480	3730
W36×170	2760	4150	2820	4240	2880	4330	2950	4430	3010	4520	3070	4610	3130	4710
	2690	4040	2740	4120	2800	4210	2850	4290	2910	4370	2960	4450	3010	4530
	2620	3930	2660	4000	2710	4070	2750	4140	2800	4210	2850	4280	2890	4350
	2540	3820	2580	3870	2610	3930	2650	3990	2690	4040	2730	4100	2770	4160
	2460	3690	2490	3740	2520	3780	2550	3830	2580	3870	2600	3910	2630	3960
	360	3550	2390	3590	2410	3620	2430	3650	2450	3690	2480	3720	2500	3760
	2210	3330	2230	3350	2250	3380	2260	3400	2280	3420	2290	3450	2310	3470
W36×160	2580	3880	2640	3970	2700	4060	2760	4150	2820	4240	2880	4320	2940	4410
	2520	3790	2570	3860	2620	3940	2670	4020	2720	4090	2780	4170	2830	4250
	2450	3690	2500	3750	2540	3820	2580	3880	2630	3950	2670	4010	2710	4080
	2380	3580	2420	3630	2450	3690	2490	3740	2530	3800	2560	3850	2600	3900
	2310	3470	2340	3510	2360	3550	2390	3600	2420	3640	2450	3680	2480	3720
	2220	3330	2240	3360	2260	3400	2280	3430	2300	3460	2320	3490	2350	3530
	2070	3110	2080	3130	2100	3160	2110	3180	2130	3200	2140	3220	2160	3240

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W36–W33

$F_y = 50$ ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	Y1[a]	ΣQ_n	Y2[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×150	1450	2180	TFL	0	2210	2200	3300	2250	3390	2310	3470	2360	3550
			2	0.235	1930	2170	3260	2220	3330	2270	3400	2310	3480
			3	0.470	1650	2140	3210	2180	3270	2220	3330	2260	3400
			4	0.705	1370	2100	3160	2130	3210	2170	3260	2200	3310
			BFL	0.940	1090	2060	3100	2090	3140	2110	3180	2140	3220
			6	4.82	819	2000	3000	2020	3030	2040	3060	2060	3090
			7	9.08	553	1880	2820	1890	2840	1910	2870	1920	2890
W36×135	1270	1910	TFL	0	1990	1960	2950	2010	3020	2060	3100	2110	3170
			2	0.198	1750	1940	2910	1980	2980	2020	3040	2070	3110
			3	0.395	1520	1910	2870	1950	2930	1990	2980	2020	3040
			4	0.593	1280	1880	2830	1910	2870	1940	2920	1980	2970
			BFL	0.790	1040	1850	2780	1870	2820	1900	2860	1930	2900
			6	4.94	770	1780	2680	1800	2710	1820	2740	1840	2770
			7	9.49	497	1660	2490	1670	2510	1680	2530	1700	2550
W33×221	2140	3210	TFL	0	3260	3080	4630	3160	4760	3250	4880	3330	5000
			2	0.319	2750	3030	4550	3100	4660	3170	4760	3230	4860
			3	0.638	2250	2970	4460	3020	4540	3080	4630	3130	4710
			4	0.956	1750	2900	4350	2940	4420	2980	4480	3030	4550
			BFL	1.28	1240	2820	4230	2850	4280	2880	4330	2910	4370
			6	3.69	1030	2770	4170	2800	4200	2820	4240	2850	4280
			7	6.46	814	2700	4050	2720	4080	2740	4110	2760	4140
W33×201	1930	2900	TFL	0	2960	2780	4180	2860	4290	2930	4400	3000	4510
			2	0.288	2510	2730	4110	2800	4200	2860	4300	2920	4390
			3	0.575	2050	2680	4030	2730	4100	2780	4180	2830	4260
			4	0.863	1600	2620	3930	2660	3990	2700	4050	2740	4110
			BFL	1.15	1150	2550	3830	2580	3870	2610	3920	2640	3960
			6	3.64	944	2510	3770	2530	3800	2550	3840	2580	3870
			7	6.49	740	2430	3660	2450	3690	2470	3710	2490	3740
W33×169	1570	2360	TFL	0	2480	2340	3510	2400	3600	2460	3700	2520	3790
			2	0.305	2120	2300	3450	2350	3530	2400	3610	2460	3690
			3	0.610	1770	2250	3390	2300	3460	2340	3520	2390	3590
			4	0.915	1420	2210	3320	2240	3370	2280	3420	2310	3480
			BFL	1.22	1070	2150	3240	2180	3280	2210	3320	2230	3360
			6	4.29	846	2100	3160	2120	3190	2140	3220	2160	3250
			7	7.67	619	2010	3020	2030	3050	2040	3070	2060	3090

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W36–W33

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W36×150	**2420**	3640	**2470**	3720	**2530**	3800	**2590**	3890	**2640**	3970	**2700**	4050	**2750**	4130
	2360	3550	**2410**	3620	**2460**	3690	**2510**	3770	**2550**	3840	**2600**	3910	**2650**	3980
	2300	3460	**2340**	3520	**2380**	3580	**2420**	3640	**2470**	3710	**2510**	3770	**2550**	3830
	2240	3360	**2270**	3410	**2300**	3460	**2340**	3520	**2370**	3570	**2410**	3620	**2440**	3670
	2170	3260	**2200**	3300	**2220**	3340	**2250**	3380	**2280**	3420	**2300**	3460	**2330**	3500
	2080	3130	**2100**	3160	**2120**	3190	**2140**	3220	**2160**	3250	**2180**	3280	**2200**	3310
	1930	2910	**1950**	2930	**1960**	2950	**1980**	2970	**1990**	2990	**2000**	3010	**2020**	3030
W36×135	**2160**	3250	**2210**	3320	**2260**	3390	**2310**	3470	**2360**	3540	**2410**	3620	**2460**	3690
	2110	3170	**2150**	3240	**2200**	3300	**2240**	3370	**2290**	3440	**2330**	3500	**2370**	3570
	2060	3100	**2100**	3150	**2140**	3210	**2170**	3270	**2210**	3320	**2250**	3380	**2290**	3440
	2010	3020	**2040**	3070	**2070**	3110	**2100**	3160	**2140**	3210	**2170**	3260	**2200**	3310
	1950	2930	**1980**	2970	**2000**	3010	**2030**	3050	**2060**	3090	**2080**	3130	**2110**	3170
	1860	2800	**1880**	2830	**1900**	2860	**1920**	2880	**1940**	2910	**1960**	2940	**1980**	2970
	1710	2570	**1720**	2590	**1730**	2610	**1750**	2620	**1760**	2640	**1770**	2660	**1780**	2680
W33×221	**3410**	5120	**3490**	5240	**3570**	5370	**3650**	5490	**3730**	5610	**3810**	5730	**3900**	5860
	3300	4970	**3370**	5070	**3440**	5170	**3510**	5270	**3580**	5380	**3650**	5480	**3720**	5580
	3190	4800	**3250**	4880	**3300**	4960	**3360**	5050	**3420**	5130	**3470**	5220	**3530**	5300
	3070	4620	**3110**	4680	**3160**	4750	**3200**	4810	**3240**	4880	**3290**	4940	**3330**	5010
	2940	4420	**2970**	4470	**3000**	4510	**3030**	4560	**3070**	4610	**3100**	4650	**3130**	4700
	2870	4320	**2900**	4360	**2930**	4400	**2950**	4440	**2980**	4480	**3000**	4510	**3030**	4550
	2780	4170	**2800**	4200	**2820**	4240	**2840**	4270	**2860**	4300	**2880**	4330	**2900**	4360
W33×201	**3080**	4630	**3150**	4740	**3220**	4850	**3300**	4960	**3370**	5070	**3450**	5180	**3520**	5290
	2980	4480	**3050**	4580	**3110**	4670	**3170**	4770	**3230**	4860	**3300**	4950	**3360**	5050
	2880	4330	**2930**	4410	**2990**	4490	**3040**	4560	**3090**	4640	**3140**	4720	**3190**	4800
	2780	4170	**2820**	4230	**2860**	4290	**2900**	4350	**2940**	4410	**2980**	4470	**3020**	4530
	2660	4000	**2690**	4050	**2720**	4090	**2750**	4130	**2780**	4180	**2810**	4220	**2840**	4260
	2600	3910	**2620**	3940	**2650**	3980	**2670**	4010	**2690**	4050	**2720**	4090	**2740**	4120
	2510	3770	**2530**	3800	**2550**	3830	**2560**	3850	**2580**	3880	**2600**	3910	**2620**	3940
W33×169	**2580**	3880	**2640**	3970	**2710**	4070	**2770**	4160	**2830**	4250	**2890**	4350	**2950**	4440
	2510	3770	**2560**	3850	**2620**	3930	**2670**	4010	**2720**	4090	**2780**	4170	**2830**	4250
	2430	3660	**2480**	3720	**2520**	3790	**2560**	3850	**2610**	3920	**2650**	3990	**2700**	4050
	2350	3530	**2380**	3580	**2420**	3640	**2460**	3690	**2490**	3740	**2530**	3800	**2560**	3850
	2260	3400	**2290**	3440	**2310**	3480	**2340**	3520	**2370**	3560	**2390**	3600	**2420**	3640
	2190	3280	**2210**	3320	**2230**	3350	**2250**	3380	**2270**	3410	**2290**	3440	**2310**	3480
	2070	3120	**2090**	3140	**2100**	3160	**2120**	3180	**2130**	3210	**2150**	3230	**2170**	3250

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W33–W30

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W33×152	**1390**	2100	TFL	0	2240	**2090**	3150	**2150**	3230	**2200**	3310	**2260**	3400
			2	0.264	1930	**2060**	3100	**2110**	3170	**2160**	3240	**2210**	3310
			3	0.528	1630	**2020**	3040	**2060**	3100	**2110**	3160	**2150**	3230
			4	0.791	1320	**1980**	2980	**2020**	3030	**2050**	3080	**2080**	3130
			BFL	1.06	1020	**1940**	2910	**1960**	2950	**1990**	2990	**2020**	3030
			6	4.33	788	**1890**	2840	**1910**	2870	**1930**	2900	**1950**	2930
			7	7.94	559	**1790**	2700	**1810**	2720	**1820**	2740	**1840**	2760
W33×141	**1280**	1930	TFL	0	2080	**1930**	2910	**1990**	2980	**2040**	3060	**2090**	3140
			2	0.240	1800	**1900**	2860	**1950**	2930	**1990**	3000	**2040**	3060
			3	0.480	1520	**1870**	2810	**1910**	2870	**1950**	2930	**1990**	2980
			4	0.720	1250	**1840**	2760	**1870**	2810	**1900**	2850	**1930**	2900
			BFL	0.960	970	**1800**	2700	**1820**	2740	**1840**	2770	**1870**	2810
			6	4.34	745	**1750**	2620	**1760**	2650	**1780**	2680	**1800**	2710
			7	8.06	519	**1650**	2490	**1670**	2510	**1680**	2530	**1690**	2540
W33×130	**1170**	1750	TFL	0	1920	**1770**	2660	**1820**	2740	**1870**	2810	**1920**	2880
			2	0.214	1670	**1750**	2620	**1790**	2690	**1830**	2750	**1870**	2810
			3	0.428	1420	**1720**	2580	**1750**	2640	**1790**	2690	**1820**	2740
			4	0.641	1180	**1690**	2540	**1720**	2580	**1750**	2620	**1780**	2670
			BFL	0.855	931	**1650**	2480	**1680**	2520	**1700**	2550	**1720**	2590
			6	4.39	705	**1600**	2410	**1620**	2440	**1640**	2460	**1660**	2490
			7	8.29	479	**1510**	2270	**1520**	2290	**1530**	2300	**1540**	2320
W33×118	**1040**	1560	TFL	0	1730	**1590**	2400	**1640**	2460	**1680**	2530	**1720**	2590
			2	0.185	1520	**1570**	2360	**1610**	2420	**1650**	2480	**1690**	2530
			3	0.370	1310	**1550**	2330	**1580**	2380	**1610**	2420	**1650**	2470
			4	0.555	1100	**1520**	2290	**1550**	2330	**1580**	2370	**1600**	2410
			BFL	0.740	884	**1490**	2250	**1520**	2280	**1540**	2310	**1560**	2340
			6	4.45	659	**1440**	2170	**1460**	2200	**1480**	2220	**1490**	2250
			7	8.55	433	**1350**	2030	**1360**	2040	**1370**	2060	**1380**	2080
W30×116	**943**	1420	TFL	0	1710	**1450**	2180	**1490**	2240	**1540**	2310	**1580**	2370
			2	0.213	1490	**1430**	2150	**1460**	2200	**1500**	2260	**1540**	2310
			3	0.425	1260	**1400**	2110	**1430**	2150	**1460**	2200	**1500**	2250
			4	0.638	1040	**1370**	2060	**1400**	2100	**1430**	2140	**1450**	2180
			BFL	0.850	818	**1340**	2020	**1360**	2050	**1380**	2080	**1400**	2110
			6	3.98	623	**1300**	1960	**1320**	1980	**1330**	2000	**1350**	2030
			7	7.44	427	**1230**	1840	**1240**	1860	**1250**	1880	**1260**	1890

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W33–W30

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
W33×152	**2320**	3480	**2370**	3570	**2430**	3650	**2480**	3730	**2540**	3820	**2600**	3900	**2650**	3980
	2250	3390	**2300**	3460	**2350**	3530	**2400**	3600	**2450**	3680	**2490**	3750	**2540**	3820
	2190	3290	**2230**	3350	**2270**	3410	**2310**	3470	**2350**	3530	**2390**	3590	**2430**	3650
	2120	3180	**2150**	3230	**2180**	3280	**2210**	3330	**2250**	3380	**2280**	3430	**2310**	3480
	2040	3070	**2070**	3110	**2090**	3140	**2120**	3180	**2140**	3220	**2170**	3260	**2190**	3300
	1970	2950	**1990**	2980	**2010**	3010	**2030**	3040	**2040**	3070	**2060**	3100	**2080**	3130
	1850	2780	**1860**	2800	**1880**	2820	**1890**	2840	**1910**	2860	**1920**	2890	**1930**	2910
W33×141	**2140**	3220	**2190**	3300	**2240**	3370	**2300**	3450	**2350**	3530	**2400**	3610	**2450**	3690
	2080	3130	**2130**	3200	**2170**	3270	**2220**	3330	**2260**	3400	**2310**	3470	**2350**	3540
	2020	3040	**2060**	3100	**2100**	3160	**2140**	3210	**2180**	3270	**2210**	3330	**2250**	3380
	1960	2950	**1990**	2990	**2020**	3040	**2050**	3090	**2080**	3130	**2120**	3180	**2150**	3230
	1890	2850	**1920**	2880	**1940**	2920	**1970**	2950	**1990**	2990	**2010**	3030	**2040**	3060
	1820	2740	**1840**	2760	**1860**	2790	**1880**	2820	**1890**	2850	**1910**	2880	**1930**	2900
	1710	2560	**1720**	2580	**1730**	2600	**1740**	2620	**1760**	2640	**1770**	2660	**1780**	2680
W33×130	**1960**	2950	**2010**	3020	**2060**	3090	**2110**	3170	**2150**	3240	**2200**	3310	**2250**	3380
	1910	2880	**1950**	2940	**2000**	3000	**2040**	3060	**2080**	3130	**2120**	3190	**2160**	3250
	1860	2800	**1900**	2850	**1930**	2900	**1970**	2960	**2000**	3010	**2040**	3060	**2070**	3120
	1800	2710	**1830**	2760	**1860**	2800	**1890**	2840	**1920**	2890	**1950**	2930	**1980**	2980
	1750	2620	**1770**	2660	**1790**	2690	**1820**	2730	**1840**	2760	**1860**	2800	**1890**	2830
	1670	2520	**1690**	2540	**1710**	2570	**1730**	2590	**1740**	2620	**1760**	2650	**1780**	2670
	1560	2340	**1570**	2360	**1580**	2380	**1590**	2390	**1600**	2410	**1620**	2430	**1630**	2450
W33×118	**1770**	2660	**1810**	2720	**1850**	2790	**1900**	2850	**1940**	2920	**1980**	2980	**2030**	3050
	1720	2590	**1760**	2650	**1800**	2710	**1840**	2760	**1880**	2820	**1910**	2880	**1950**	2930
	1680	2520	**1710**	2570	**1740**	2620	**1780**	2670	**1810**	2720	**1840**	2770	**1870**	2820
	1630	2450	**1660**	2490	**1690**	2530	**1710**	2580	**1740**	2620	**1770**	2660	**1800**	2700
	1580	2380	**1600**	2410	**1630**	2440	**1650**	2480	**1670**	2510	**1690**	2540	**1710**	2580
	1510	2270	**1530**	2290	**1540**	2320	**1560**	2340	**1580**	2370	**1590**	2390	**1610**	2420
	1390	2090	**1400**	2110	**1410**	2120	**1420**	2140	**1440**	2160	**1450**	2170	**1460**	2190
W30×116	**1620**	2440	**1660**	2500	**1710**	2570	**1750**	2630	**1790**	2690	**1830**	2760	**1880**	2820
	1580	2370	**1610**	2420	**1650**	2480	**1690**	2540	**1720**	2590	**1760**	2650	**1800**	2700
	1530	2300	**1560**	2340	**1590**	2390	**1620**	2440	**1650**	2490	**1690**	2530	**1720**	2580
	1480	2220	**1500**	2260	**1530**	2300	**1560**	2340	**1580**	2380	**1610**	2420	**1630**	2450
	1420	2140	**1450**	2170	**1470**	2200	**1490**	2230	**1510**	2260	**1530**	2290	**1550**	2330
	1360	2050	**1380**	2070	**1390**	2100	**1410**	2120	**1430**	2140	**1440**	2170	**1460**	2190
	1270	1910	**1280**	1920	**1290**	1940	**1300**	1960	**1310**	1970	**1320**	1990	**1330**	2000

ASD	**LRFD**
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

W30–W27

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W30×108	863	1300	TFL	0	1590	1340	2010	1380	2070	1420	2130	1460	2190
			2	0.190	1390	1320	1980	1350	2030	1390	2080	1420	2140
			3	0.380	1190	1290	1950	1320	1990	1350	2040	1380	2080
			4	0.570	989	1270	1910	1300	1950	1320	1980	1340	2020
			BFL	0.760	790	1240	1870	1260	1900	1280	1930	1300	1960
			6	4.03	593	1200	1810	1220	1830	1230	1850	1250	1870
			7	7.64	396	1130	1690	1140	1710	1150	1720	1160	1740
W30×99	778	1170	TFL	0	1450	1220	1840	1260	1890	1290	1940	1330	2000
			2	0.168	1280	1200	1810	1230	1860	1270	1900	1300	1950
			3	0.335	1100	1180	1780	1210	1820	1240	1860	1270	1900
			4	0.503	929	1160	1750	1190	1780	1210	1820	1230	1850
			BFL	0.670	754	1140	1710	1160	1740	1180	1770	1200	1800
			6	4.08	559	1100	1650	1110	1670	1130	1690	1140	1710
			7	7.83	364	1020	1530	1030	1550	1040	1560	1050	1580
W30×90	706	1060	TFL	0	1320	1100	1660	1140	1710	1170	1760	1200	1810
			2	0.153	1160	1090	1630	1110	1680	1140	1720	1170	1760
			3	0.305	1000	1070	1610	1090	1640	1120	1680	1140	1720
			4	0.458	842	1050	1580	1070	1610	1090	1640	1110	1670
			BFL	0.610	683	1030	1550	1050	1570	1060	1600	1080	1620
			6	3.99	506	993	1490	1010	1510	1020	1530	1030	1550
			7	7.76	329	924	1390	932	1400	940	1410	948	1430
W27×102	761	1140	TFL	0	1500	1160	1750	1200	1810	1240	1860	1280	1920
			2	0.208	1290	1140	1720	1180	1770	1210	1810	1240	1860
			3	0.415	1090	1120	1680	1150	1720	1170	1760	1200	1800
			4	0.623	879	1090	1640	1110	1680	1140	1710	1160	1740
			BFL	0.830	671	1060	1600	1080	1630	1100	1650	1110	1680
			6	3.38	523	1040	1560	1050	1580	1060	1600	1070	1620
			7	6.26	375	985	1480	994	1490	1000	1510	1010	1520
W27×94	694	1040	TFL	0	1380	1070	1600	1100	1660	1140	1710	1170	1760
			2	0.186	1200	1050	1570	1080	1620	1110	1660	1140	1710
			3	0.373	1010	1030	1540	1050	1580	1080	1620	1100	1660
			4	0.559	824	1000	1510	1020	1540	1040	1570	1060	1600
			BFL	0.745	638	978	1470	994	1490	1010	1520	1030	1540
			6	3.43	492	950	1430	962	1450	974	1460	987	1480
			7	6.41	346	899	1350	908	1360	917	1380	925	1390

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W30–W27

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W30×108	**1500**	2250	**1540**	2310	**1580**	2370	**1620**	2430	**1650**	2490	**1690**	2550	**1730**	2610
	1460	2190	**1490**	2240	**1530**	2290	**1560**	2340	**1590**	2400	**1630**	2450	**1660**	2500
	1410	2120	**1440**	2170	**1470**	2210	**1500**	2260	**1530**	2300	**1560**	2350	**1590**	2390
	1370	2060	**1390**	2090	**1420**	2130	**1440**	2170	**1470**	2210	**1490**	2240	**1520**	2280
	1320	1990	**1340**	2020	**1360**	2050	**1380**	2080	**1400**	2110	**1420**	2140	**1440**	2170
	1260	1900	**1280**	1920	**1290**	1940	**1310**	1960	**1320**	1990	**1340**	2010	**1350**	2030
	1170	1750	**1170**	1770	**1180**	1780	**1190**	1800	**1200**	1810	**1210**	1830	**1220**	1840
W30×99	**1370**	2050	**1400**	2110	**1440**	2160	**1480**	2220	**1510**	2270	**1550**	2330	**1580**	2380
	1330	2000	**1360**	2050	**1390**	2100	**1430**	2140	**1460**	2190	**1490**	2240	**1520**	2290
	1290	1940	**1320**	1990	**1350**	2030	**1380**	2070	**1400**	2110	**1430**	2150	**1460**	2190
	1250	1890	**1280**	1920	**1300**	1960	**1320**	1990	**1350**	2030	**1370**	2060	**1390**	2090
	1210	1830	**1230**	1850	**1250**	1880	**1270**	1910	**1290**	1940	**1310**	1970	**1330**	2000
	1150	1730	**1170**	1760	**1180**	1780	**1200**	1800	**1210**	1820	**1220**	1840	**1240**	1860
	1060	1590	**1070**	1600	**1080**	1620	**1080**	1630	**1090**	1640	**1100**	1660	**1110**	1670
W30×90	**1230**	1850	**1270**	1900	**1300**	1950	**1330**	2000	**1370**	2050	**1400**	2100	**1430**	2150
	1200	1810	**1230**	1850	**1260**	1890	**1290**	1940	**1320**	1980	**1350**	2020	**1380**	2070
	1170	1760	**1190**	1790	**1220**	1830	**1240**	1870	**1270**	1910	**1290**	1940	**1320**	1980
	1130	1700	**1150**	1740	**1180**	1770	**1200**	1800	**1220**	1830	**1240**	1860	**1260**	1890
	1100	1650	**1110**	1680	**1130**	1700	**1150**	1730	**1170**	1750	**1180**	1780	**1200**	1800
	1040	1570	**1060**	1590	**1070**	1610	**1080**	1630	**1090**	1640	**1110**	1660	**1120**	1680
	956	1440	**965**	1450	**973**	1460	**981**	1470	**989**	1490	**997**	1500	**1010**	1510
W27×102	**1310**	1980	**1350**	2030	**1390**	2090	**1430**	2140	**1460**	2200	**1500**	2260	**1540**	2310
	1270	1910	**1300**	1960	**1340**	2010	**1370**	2060	**1400**	2110	**1430**	2150	**1470**	2200
	1230	1840	**1250**	1890	**1280**	1930	**1310**	1970	**1340**	2010	**1360**	2050	**1390**	2090
	1180	1770	**1200**	1810	**1220**	1840	**1250**	1870	**1270**	1910	**1290**	1940	**1310**	1970
	1130	1700	**1150**	1730	**1170**	1750	**1180**	1780	**1200**	1800	**1220**	1830	**1230**	1850
	1090	1630	**1100**	1650	**1110**	1670	**1130**	1690	**1140**	1710	**1150**	1730	**1170**	1750
	1020	1540	**1030**	1550	**1040**	1560	**1050**	1580	**1060**	1590	**1070**	1610	**1080**	1620
W27×94	**1200**	1810	**1240**	1860	**1270**	1910	**1310**	1970	**1340**	2020	**1380**	2070	**1410**	2120
	1170	1750	**1200**	1800	**1230**	1840	**1260**	1890	**1290**	1930	**1320**	1980	**1350**	2020
	1130	1690	**1150**	1730	**1180**	1770	**1200**	1810	**1230**	1850	**1250**	1880	**1280**	1920
	1090	1630	**1110**	1660	**1130**	1690	**1150**	1720	**1170**	1750	**1190**	1790	**1210**	1820
	1040	1570	**1060**	1590	**1070**	1610	**1090**	1640	**1110**	1660	**1120**	1690	**1140**	1710
	999	1500	**1010**	1520	**1020**	1540	**1040**	1560	**1050**	1580	**1060**	1590	**1070**	1610
	934	1400	**943**	1420	**951**	1430	**960**	1440	**968**	1460	**977**	1470	**986**	1480

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W27–W24

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W27×84	**609**	915	TFL	0	1240	**948**	1430	**979**	1470	**1010**	1520	**1040**	1560
			2	0.160	1080	**932**	1400	**959**	1440	**986**	1480	**1010**	1520
			3	0.320	918	**914**	1370	**937**	1410	**960**	1440	**983**	1480
			4	0.480	758	**895**	1340	**914**	1370	**932**	1400	**951**	1430
			BFL	0.640	598	**874**	1310	**889**	1340	**904**	1360	**919**	1380
			6	3.50	454	**846**	1270	**857**	1290	**869**	1310	**880**	1320
			7	6.63	309	**795**	1200	**803**	1210	**811**	1220	**818**	1230
W24×94	**634**	953	TFL	0	1380	**978**	1470	**1010**	1520	**1050**	1570	**1080**	1630
			2	0.219	1190	**957**	1440	**987**	1480	**1020**	1530	**1050**	1570
			3	0.438	988	**934**	1400	**959**	1440	**984**	1480	**1010**	1520
			4	0.656	790	**909**	1370	**929**	1400	**948**	1430	**968**	1460
			BFL	0.875	592	**882**	1330	**896**	1350	**911**	1370	**926**	1390
			6	3.05	469	**858**	1290	**870**	1310	**882**	1320	**893**	1340
			7	5.43	346	**820**	1230	**829**	1250	**837**	1260	**846**	1270
W24×84	**559**	840	TFL	0	1240	**866**	1300	**897**	1350	**928**	1390	**959**	1440
			2	0.193	1060	**848**	1270	**874**	1310	**901**	1350	**927**	1390
			3	0.385	888	**828**	1240	**850**	1280	**872**	1310	**895**	1340
			4	0.578	715	**807**	1210	**825**	1240	**842**	1270	**860**	1290
			BFL	0.770	541	**784**	1180	**797**	1200	**810**	1220	**824**	1240
			6	3.01	425	**762**	1140	**772**	1160	**783**	1180	**793**	1190
			7	5.48	309	**726**	1090	**733**	1100	**741**	1110	**749**	1130
W24×76	**499**	750	TFL	0	1120	**779**	1170	**807**	1210	**835**	1250	**863**	1300
			2	0.170	966	**763**	1150	**787**	1180	**811**	1220	**835**	1260
			3	0.340	813	**746**	1120	**766**	1150	**787**	1180	**807**	1210
			4	0.510	660	**727**	1090	**744**	1120	**760**	1140	**777**	1170
			BFL	0.680	507	**708**	1060	**720**	1080	**733**	1100	**746**	1120
			6	3.02	393	**687**	1030	**696**	1050	**706**	1060	**716**	1080
			7	5.61	280	**651**	978	**658**	988	**665**	999	**672**	1010
W24×68	**442**	664	TFL	0	1000	**694**	1040	**719**	1080	**744**	1120	**769**	1160
			2	0.146	872	**681**	1020	**702**	1060	**724**	1090	**746**	1120
			3	0.293	741	**666**	1000	**685**	1030	**703**	1060	**722**	1080
			4	0.439	610	**651**	978	**666**	1000	**681**	1020	**696**	1050
			BFL	0.585	479	**634**	953	**646**	971	**658**	989	**670**	1010
			6	3.07	365	**613**	922	**622**	935	**631**	949	**640**	963
			7	5.82	251	**576**	866	**583**	876	**589**	885	**595**	895

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W27–W24

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W27×84	1070	1610	1100	1660	1130	1700	1160	1750	1200	1800	1230	1840	1260	1890
	1040	1560	1070	1600	1090	1640	1120	1680	1150	1720	1170	1760	1200	1800
	1010	1510	1030	1550	1050	1580	1070	1610	1100	1650	1120	1680	1140	1720
	970	1460	989	1490	1010	1520	1030	1540	1050	1570	1060	1600	1080	1630
	934	1400	949	1430	964	1450	979	1470	994	1490	1010	1520	1020	1540
	891	1340	903	1360	914	1370	925	1390	937	1410	948	1420	959	1440
	826	1240	834	1250	841	1260	849	1280	857	1290	865	1300	872	1310
W24×94	1120	1680	1150	1730	1190	1780	1220	1830	1250	1890	1290	1940	1320	1990
	1080	1620	1110	1660	1130	1710	1160	1750	1190	1790	1220	1840	1250	1880
	1030	1550	1060	1590	1080	1630	1110	1660	1130	1700	1160	1740	1180	1770
	988	1480	1010	1510	1030	1540	1050	1570	1070	1600	1090	1630	1110	1660
	941	1410	956	1440	970	1460	985	1480	1000	1500	1010	1520	1030	1550
	905	1360	917	1380	928	1400	940	1410	952	1430	963	1450	975	1470
	854	1280	863	1300	872	1310	880	1320	889	1340	898	1350	906	1360
W24×84	989	1490	1020	1530	1050	1580	1080	1630	1110	1670	1140	1720	1170	1770
	954	1430	980	1470	1010	1510	1030	1550	1060	1590	1090	1630	1110	1670
	917	1380	939	1410	961	1440	983	1480	1010	1510	1030	1540	1050	1580
	878	1320	896	1350	914	1370	931	1400	949	1430	967	1450	985	1480
	837	1260	851	1280	864	1300	878	1320	891	1340	905	1360	918	1380
	804	1210	815	1220	825	1240	836	1260	847	1270	857	1290	868	1300
	756	1140	764	1150	772	1160	779	1170	787	1180	795	1190	803	1210
W24×76	891	1340	919	1380	946	1420	974	1460	1000	1510	1030	1550	1060	1590
	860	1290	884	1330	908	1360	932	1400	956	1440	980	1470	1000	1510
	827	1240	847	1270	868	1300	888	1330	908	1370	928	1400	949	1430
	793	1190	810	1220	826	1240	843	1270	859	1290	876	1320	892	1340
	758	1140	771	1160	784	1180	796	1200	809	1220	822	1230	834	1250
	726	1090	736	1110	745	1120	755	1140	765	1150	775	1160	785	1180
	679	1020	686	1030	693	1040	700	1050	707	1060	714	1070	721	1080
W24×68	794	1190	819	1230	844	1270	869	1310	894	1340	919	1380	944	1420
	768	1150	789	1190	811	1220	833	1250	855	1280	876	1320	898	1350
	740	1110	759	1140	777	1170	795	1200	814	1220	832	1250	851	1280
	711	1070	727	1090	742	1120	757	1140	772	1160	788	1180	803	1210
	682	1020	694	1040	706	1060	718	1080	730	1100	742	1110	754	1130
	650	976	659	990	668	1000	677	1020	686	1030	695	1040	704	1060
	602	904	608	913	614	923	620	932	627	942	633	951	639	961

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W24–W21

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W24×62	382	574	TFL	0	911	631	948	653	982	676	1020	699	1050
			2	0.148	807	620	932	640	962	660	992	680	1020
			3	0.295	703	608	914	626	941	643	967	661	993
			4	0.443	600	596	896	611	918	626	941	641	963
			BFL	0.590	496	583	876	595	895	608	914	620	932
			6	3.46	362	557	837	566	851	575	864	584	878
			7	6.57	228	510	767	516	775	522	784	527	792
W24×55	334	503	TFL	0	810	557	837	577	868	598	898	618	929
			2	0.126	722	548	824	566	851	584	878	602	905
			3	0.253	633	538	809	554	833	570	857	586	881
			4	0.379	545	528	794	542	814	555	835	569	855
			BFL	0.505	456	517	778	529	795	540	812	552	829
			6	3.45	329	493	741	501	753	510	766	518	778
			7	6.66	203	449	674	454	682	459	689	464	697
W21×73	429	645	TFL	0	1070	676	1020	703	1060	730	1100	757	1140
			2	0.185	921	660	993	683	1030	706	1060	729	1100
			3	0.370	767	643	966	662	995	681	1020	700	1050
			4	0.555	614	624	938	639	961	655	984	670	1010
			BFL	0.740	460	604	908	615	925	627	942	638	959
			6	2.61	364	587	882	596	896	605	909	614	923
			7	4.72	269	560	841	566	851	573	861	580	872
W21×68	399	600	TFL	0	1000	628	944	653	982	678	1020	703	1060
			2	0.171	860	614	922	635	954	656	987	678	1020
			3	0.343	719	598	898	616	925	633	952	651	979
			4	0.514	577	580	872	595	894	609	916	624	937
			BFL	0.685	436	562	845	573	861	584	878	595	894
			6	2.59	343	546	820	554	833	563	846	572	859
			7	4.74	251	520	781	526	791	532	800	539	809
W21×62	359	540	TFL	0	913	569	855	592	890	615	924	637	958
			2	0.154	786	556	836	576	865	595	895	615	924
			3	0.308	659	542	814	558	839	575	864	591	889
			4	0.461	533	527	792	540	812	553	832	567	852
			BFL	0.615	406	511	768	521	783	531	798	541	813
			6	2.57	317	495	745	503	756	511	768	519	780
			7	4.79	228	470	707	476	715	482	724	487	732

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

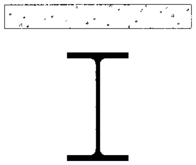
[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft



W24–W21

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
W24×62	**722**	1080	**744**	1120	**767**	1150	**790**	1190	**812**	1220	**835**	1260	**858**	1290
	700	1050	**721**	1080	**741**	1110	**761**	1140	**781**	1170	**801**	1200	**821**	1230
	679	1020	**696**	1050	**714**	1070	**731**	1100	**749**	1130	**766**	1150	**784**	1180
	656	986	**671**	1010	**686**	1030	**701**	1050	**716**	1080	**731**	1100	**746**	1120
	633	951	**645**	969	**657**	988	**670**	1010	**682**	1030	**694**	1040	**707**	1060
	593	891	**602**	905	**611**	919	**620**	932	**629**	946	**638**	959	**647**	973
	533	801	**539**	810	**544**	818	**550**	827	**556**	835	**561**	844	**567**	852
W24×55	**638**	959	**658**	989	**678**	1020	**699**	1050	**719**	1080	**739**	1110	**759**	1140
	620	932	**638**	959	**656**	986	**674**	1010	**692**	1040	**710**	1070	**728**	1090
	602	904	**617**	928	**633**	952	**649**	975	**665**	999	**681**	1020	**696**	1050
	583	876	**596**	896	**610**	917	**623**	937	**637**	957	**651**	978	**664**	998
	563	846	**574**	863	**586**	880	**597**	897	**608**	915	**620**	932	**631**	949
	526	790	**534**	803	**542**	815	**551**	827	**559**	840	**567**	852	**575**	865
	469	705	**474**	712	**479**	720	**484**	727	**489**	735	**494**	743	**499**	750
W21×73	**784**	1180	**810**	1220	**837**	1260	**864**	1300	**891**	1340	**918**	1380	**944**	1420
	752	1130	**775**	1170	**798**	1200	**821**	1230	**844**	1270	**867**	1300	**890**	1340
	719	1080	**739**	1110	**758**	1140	**777**	1170	**796**	1200	**815**	1230	**834**	1250
	685	1030	**701**	1050	**716**	1080	**731**	1100	**747**	1120	**762**	1150	**777**	1170
	650	977	**661**	994	**673**	1010	**684**	1030	**696**	1050	**707**	1060	**719**	1080
	623	937	**632**	950	**641**	964	**650**	978	**660**	991	**669**	1000	**678**	1020
	587	882	**593**	892	**600**	902	**607**	912	**613**	922	**620**	932	**627**	942
W21×68	**728**	1090	**753**	1130	**778**	1170	**803**	1210	**828**	1240	**853**	1280	**878**	1320
	699	1050	**721**	1080	**742**	1120	**764**	1150	**785**	1180	**807**	1210	**828**	1240
	669	1010	**687**	1030	**705**	1060	**723**	1090	**741**	1110	**759**	1140	**777**	1170
	638	959	**652**	981	**667**	1000	**681**	1020	**696**	1050	**710**	1070	**724**	1090
	606	910	**616**	927	**627**	943	**638**	959	**649**	976	**660**	992	**671**	1010
	580	872	**589**	885	**597**	898	**606**	910	**614**	923	**623**	936	**631**	949
	545	819	**551**	828	**557**	838	**564**	847	**570**	856	**576**	866	**582**	875
W21×62	**660**	992	**683**	1030	**706**	1060	**728**	1090	**751**	1130	**774**	1160	**797**	1200
	634	953	**654**	983	**674**	1010	**693**	1040	**713**	1070	**732**	1100	**752**	1130
	608	913	**624**	938	**641**	963	**657**	987	**673**	1010	**690**	1040	**706**	1060
	580	872	**593**	892	**607**	912	**620**	932	**633**	952	**646**	972	**660**	992
	551	829	**561**	844	**571**	859	**582**	874	**592**	889	**602**	905	**612**	920
	527	792	**535**	804	**543**	816	**551**	828	**559**	840	**567**	852	**574**	863
	493	741	**499**	749	**504**	758	**510**	767	**516**	775	**521**	784	**527**	792

ASD	**LRFD**
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

W21

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W21×57	322	484	TFL	0	837	523	787	544	818	565	849	586	881
			2	0.163	730	512	770	530	797	549	825	567	852
			3	0.325	624	500	752	516	775	531	799	547	822
			4	0.488	517	488	733	500	752	513	772	526	791
			BFL	0.650	411	474	712	484	728	494	743	505	758
			6	2.87	310	456	685	463	696	471	708	479	720
			7	5.36	209	425	639	430	646	435	654	441	662
W21×55	314	473	TFL	0	810	501	754	522	784	542	814	562	845
			2	0.131	703	490	737	508	763	525	790	543	816
			3	0.261	596	479	719	493	742	508	764	523	786
			4	0.392	488	466	701	478	719	490	737	503	755
			BFL	0.522	381	453	681	462	695	472	709	481	724
			6	2.62	292	438	658	445	669	452	680	459	691
			7	5.00	203	412	619	417	627	422	634	427	642
W21×50	274	413	TFL	0	736	456	685	474	712	492	740	511	768
			2	0.134	648	447	671	463	696	479	720	495	744
			3	0.268	561	437	657	451	678	465	699	479	720
			4	0.401	474	427	642	439	659	451	677	462	695
			BFL	0.535	386	416	625	426	640	435	654	445	669
			6	2.91	285	398	598	405	609	412	620	419	630
			7	5.58	184	366	551	371	558	376	565	380	572
W21×48	267	401	TFL	0	707	434	652	452	679	469	705	487	732
			2	0.108	619	425	639	440	662	456	685	471	708
			3	0.215	532	416	625	429	645	442	665	455	684
			4	0.323	444	406	610	417	626	428	643	439	660
			BFL	0.430	357	395	594	404	608	413	621	422	634
			6	2.69	267	380	571	387	581	393	591	400	601
			7	5.26	177	353	531	358	538	362	544	366	551
W21×44	238	358	TFL	0	649	399	600	416	625	432	649	448	673
			2	0.113	576	392	589	406	611	421	632	435	654
			3	0.225	503	384	577	396	596	409	615	422	634
			4	0.338	430	376	565	386	581	397	597	408	613
			BFL	0.450	357	367	551	376	565	385	578	394	592
			6	2.92	259	350	526	356	535	363	545	369	555
			7	5.69	162	319	480	323	486	327	492	331	498

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)

Composite W Shapes

Available Strength in Flexure, kip-ft

W21

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
W21×57	**607**	912	**628**	943	**649**	975	**669**	1010	**690**	1040	**711**	1070	**732**	1100
	585	879	**603**	907	**622**	934	**640**	962	**658**	989	**676**	1020	**694**	1040
	563	846	**578**	869	**594**	892	**609**	916	**625**	939	**640**	963	**656**	986
	539	810	**552**	830	**565**	849	**578**	869	**591**	888	**604**	907	**617**	927
	515	774	**525**	789	**535**	805	**546**	820	**556**	836	**566**	851	**576**	866
	487	731	**494**	743	**502**	755	**510**	766	**518**	778	**525**	790	**533**	801
	446	670	**451**	678	**456**	686	**461**	693	**467**	701	**472**	709	**477**	717
W21×55	**582**	875	**602**	905	**623**	936	**643**	966	**663**	997	**683**	1030	**704**	1060
	560	842	**578**	869	**596**	895	**613**	921	**631**	948	**648**	974	**666**	1000
	538	809	**553**	831	**568**	853	**583**	876	**597**	898	**612**	920	**627**	943
	515	774	**527**	792	**539**	810	**551**	829	**564**	847	**576**	865	**588**	884
	491	738	**500**	752	**510**	766	**519**	781	**529**	795	**539**	809	**548**	824
	467	702	**474**	713	**481**	723	**489**	734	**496**	745	**503**	756	**510**	767
	432	649	**437**	657	**442**	665	**447**	672	**452**	680	**457**	687	**462**	695
W21×50	**529**	795	**547**	823	**566**	850	**584**	878	**602**	906	**621**	933	**639**	961
	511	769	**528**	793	**544**	817	**560**	841	**576**	866	**592**	890	**608**	914
	493	741	**507**	762	**521**	783	**535**	804	**549**	825	**563**	846	**577**	867
	474	713	**486**	730	**498**	748	**510**	766	**521**	784	**533**	801	**545**	819
	455	683	**464**	698	**474**	712	**484**	727	**493**	741	**503**	756	**512**	770
	427	641	**434**	652	**441**	662	**448**	673	**455**	684	**462**	695	**469**	705
	385	578	**389**	585	**394**	592	**399**	599	**403**	606	**408**	613	**412**	620
W21×48	**505**	758	**522**	785	**540**	811	**557**	838	**575**	864	**593**	891	**610**	917
	487	732	**502**	755	**518**	778	**533**	801	**549**	825	**564**	848	**579**	871
	469	704	**482**	724	**495**	744	**508**	764	**522**	784	**535**	804	**548**	824
	450	676	**461**	693	**472**	710	**483**	726	**494**	743	**505**	760	**516**	776
	431	648	**440**	661	**449**	674	**458**	688	**467**	701	**475**	715	**484**	728
	407	611	**413**	621	**420**	631	**427**	641	**433**	651	**440**	661	**447**	671
	371	557	**375**	564	**380**	571	**384**	577	**389**	584	**393**	591	**397**	597
W21×44	**464**	698	**480**	722	**496**	746	**513**	771	**529**	795	**545**	819	**561**	844
	449	675	**464**	697	**478**	718	**492**	740	**507**	762	**521**	783	**536**	805
	434	652	**447**	671	**459**	690	**472**	709	**484**	728	**497**	747	**509**	766
	418	629	**429**	645	**440**	661	**451**	677	**461**	693	**472**	710	**483**	726
	402	605	**411**	618	**420**	632	**429**	645	**438**	658	**447**	672	**456**	685
	376	564	**382**	574	**388**	584	**395**	594	**401**	603	**408**	613	**414**	623
	335	504	**339**	510	**343**	516	**347**	522	**352**	528	**356**	534	**360**	541

ASD	**LRFD**
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W18

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×60	307	461	TFL	0	882	489	735	511	768	533	801	555	834
			2	0.174	750	476	715	494	743	513	771	532	799
			3	0.348	619	461	692	476	716	492	739	507	762
			4	0.521	488	445	669	457	687	469	705	481	723
			BFL	0.695	357	428	643	437	656	446	670	454	683
			6	2.17	288	416	626	424	637	431	647	438	658
			7	3.81	220	399	600	405	609	410	617	416	625
W18×55	279	420	TFL	0	810	447	672	467	702	487	732	507	763
			2	0.158	691	434	653	452	679	469	705	486	731
			3	0.315	573	421	633	436	655	450	676	464	698
			4	0.473	454	407	612	418	629	430	646	441	663
			BFL	0.630	336	392	589	400	602	409	614	417	627
			6	2.16	269	381	573	388	583	394	593	401	603
			7	3.86	202	364	548	369	555	374	563	380	570
W18×50	252	379	TFL	0	733	402	605	421	632	439	660	457	687
			2	0.143	626	391	588	407	612	423	635	438	659
			3	0.285	520	379	570	392	590	405	609	418	629
			4	0.428	413	367	551	377	567	388	582	398	598
			BFL	0.570	306	354	531	361	543	369	554	376	566
			6	2.10	245	344	517	350	526	356	535	362	544
			7	3.83	183	328	494	333	501	338	507	342	514
W18×46	226	340	TFL	0	677	373	560	389	585	406	611	423	636
			2	0.151	585	363	546	378	568	392	590	407	612
			3	0.303	494	353	530	365	549	378	567	390	586
			4	0.454	402	342	514	352	529	362	544	372	559
			BFL	0.605	310	330	497	338	508	346	520	354	532
			6	2.37	240	319	479	325	488	330	497	336	506
			7	4.33	169	300	450	304	457	308	463	312	469
W18×40	196	294	TFL	0	588	321	483	336	505	351	527	365	549
			2	0.131	509	313	471	326	490	339	509	351	528
			3	0.263	430	305	458	315	474	326	490	337	506
			4	0.394	351	295	444	304	457	313	470	322	483
			BFL	0.525	272	286	429	292	440	299	450	306	460
			6	2.29	210	275	414	281	422	286	430	291	438
			7	4.28	147	259	389	263	395	266	400	270	406

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W18

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×60	577	868	599	901	621	934	643	967	665	1000	687	1030	709	1070
	550	827	569	855	588	884	607	912	625	940	644	968	663	996
	523	785	538	809	553	832	569	855	584	878	600	901	615	925
	493	742	506	760	518	778	530	797	542	815	554	833	567	851
	463	696	472	710	481	723	490	737	499	750	508	763	517	777
	445	669	452	680	459	691	467	701	474	712	481	723	488	734
	421	633	427	642	432	650	438	658	443	666	449	675	454	683
W18×55	528	793	548	823	568	854	588	884	608	914	629	945	649	975
	503	757	521	783	538	809	555	834	572	860	590	886	607	912
	478	719	493	741	507	762	521	783	536	805	550	826	564	848
	452	680	464	697	475	714	486	731	498	748	509	765	520	782
	426	640	434	652	442	665	451	677	459	690	467	702	476	715
	408	613	415	623	421	633	428	643	435	653	441	663	448	674
	385	578	390	586	395	593	400	601	405	608	410	616	415	624
W18×50	475	715	494	742	512	770	530	797	549	825	567	852	585	880
	454	682	469	706	485	729	501	752	516	776	532	799	548	823
	431	648	444	668	457	687	470	707	483	726	496	746	509	765
	408	613	418	629	429	644	439	660	449	675	460	691	470	706
	384	577	392	589	399	600	407	612	415	623	422	635	430	646
	368	553	374	563	380	572	386	581	393	590	399	599	405	608
	347	521	351	528	356	535	360	542	365	549	370	555	374	562
W18×46	440	661	457	687	474	712	491	738	508	763	525	788	541	814
	421	633	436	655	451	677	465	699	480	721	494	743	509	765
	402	604	414	623	427	641	439	660	451	678	464	697	476	715
	382	574	392	589	402	604	412	620	422	635	432	650	442	665
	361	543	369	555	377	566	385	578	392	590	400	601	408	613
	342	515	348	524	354	533	360	542	366	551	372	560	378	569
	317	476	321	482	325	488	329	495	333	501	338	507	342	514
W18×40	380	571	395	593	409	615	424	637	439	659	453	681	468	704
	364	547	377	566	389	585	402	604	415	623	428	643	440	662
	347	522	358	538	369	555	380	571	390	587	401	603	412	619
	330	497	339	510	348	523	357	536	365	549	374	562	383	576
	313	470	320	480	326	491	333	501	340	511	347	521	354	531
	296	445	302	453	307	461	312	469	317	477	323	485	328	493
	274	411	277	417	281	422	285	428	288	433	292	439	296	444

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

W18–W16

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×35	**166**	249	TFL	0	515	**279**	419	**292**	438	**304**	457	**317**	477
			2	0.106	451	**272**	409	**283**	426	**295**	443	**306**	460
			3	0.213	387	**265**	399	**275**	413	**285**	428	**294**	442
			4	0.319	323	**258**	388	**266**	400	**274**	412	**282**	424
			BFL	0.425	260	**251**	377	**257**	386	**263**	396	**270**	406
			6	2.38	194	**240**	360	**245**	368	**249**	375	**254**	382
			7	4.56	129	**222**	334	**225**	338	**228**	343	**232**	348
W16×45	**205**	309	TFL	0	663	**333**	501	**350**	525	**366**	550	**383**	575
			2	0.141	564	**323**	485	**337**	506	**351**	527	**365**	549
			3	0.283	464	**312**	469	**323**	486	**335**	504	**347**	521
			4	0.424	365	**300**	451	**309**	465	**318**	479	**327**	492
			BFL	0.565	266	**288**	433	**294**	443	**301**	453	**308**	462
			6	1.81	216	**280**	421	**286**	429	**291**	437	**296**	445
			7	3.26	166	**269**	404	**273**	410	**277**	416	**281**	423
W16×40	**182**	274	TFL	0	589	**294**	442	**309**	464	**323**	486	**338**	508
			2	0.126	501	**285**	428	**297**	447	**310**	466	**322**	485
			3	0.253	412	**275**	414	**286**	429	**296**	445	**306**	460
			4	0.379	324	**265**	398	**273**	411	**281**	423	**289**	435
			BFL	0.505	236	**254**	382	**260**	391	**266**	400	**272**	409
			6	1.73	191	**248**	372	**252**	379	**257**	387	**262**	394
			7	3.18	147	**238**	357	**242**	363	**245**	368	**249**	374
W16×36	**160**	240	TFL	0	529	**262**	394	**275**	413	**288**	433	**301**	453
			2	0.108	453	**254**	382	**266**	399	**277**	416	**288**	433
			3	0.215	378	**246**	370	**256**	384	**265**	398	**274**	412
			4	0.323	303	**238**	357	**245**	368	**253**	380	**260**	391
			BFL	0.430	228	**229**	344	**234**	352	**240**	361	**246**	369
			6	1.82	180	**221**	333	**226**	340	**230**	346	**235**	353
			7	3.45	132	**210**	316	**214**	321	**217**	326	**220**	331
W16×31	**135**	203	TFL	0	456	**226**	340	**238**	357	**249**	374	**260**	391
			2	0.110	396	**220**	331	**230**	346	**240**	360	**250**	375
			3	0.220	335	**214**	321	**222**	333	**230**	346	**239**	359
			4	0.330	274	**207**	311	**213**	321	**220**	331	**227**	341
			BFL	0.440	213	**199**	300	**205**	308	**210**	316	**215**	324
			6	1.99	164	**192**	288	**196**	294	**200**	300	**204**	307
			7	3.79	114	**180**	270	**182**	274	**185**	279	**188**	283

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)

Composite W Shapes

Available Strength in Flexure, kip-ft

W18–W16

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W18×35	330	496	343	515	356	535	369	554	381	573	394	593	407	612
	317	477	328	494	340	510	351	527	362	544	373	561	385	578
	304	457	314	471	323	486	333	500	343	515	352	529	362	544
	290	436	298	448	306	461	315	473	323	485	331	497	339	509
	276	415	283	425	289	435	296	445	302	454	309	464	315	474
	259	389	264	397	269	404	274	411	278	419	283	426	288	433
	235	353	238	358	241	363	244	367	248	372	251	377	254	382
W16×45	399	600	416	625	432	650	449	675	465	700	482	724	499	749
	379	570	393	591	407	612	421	633	435	654	449	675	463	697
	358	538	370	556	381	573	393	591	405	608	416	625	428	643
	337	506	346	520	355	533	364	547	373	561	382	574	391	588
	314	472	321	482	328	492	334	502	341	512	348	522	354	532
	302	453	307	462	312	470	318	478	323	486	329	494	334	502
	285	429	290	435	294	441	298	448	302	454	306	460	310	466
W16×40	353	530	367	552	382	574	397	597	412	619	426	641	441	663
	335	503	347	522	360	541	372	560	385	578	397	597	410	616
	316	476	327	491	337	507	347	522	358	537	368	553	378	568
	297	447	306	459	314	471	322	483	330	496	338	508	346	520
	278	418	284	426	290	435	296	444	301	453	307	462	313	471
	267	401	272	408	276	415	281	422	286	430	291	437	295	444
	253	380	256	385	260	391	264	396	267	402	271	407	275	413
W16×36	315	473	328	493	341	513	354	532	367	552	381	572	394	592
	299	450	311	467	322	484	333	501	345	518	356	535	367	552
	284	427	293	441	303	455	312	469	322	483	331	498	340	512
	268	403	275	414	283	425	291	437	298	448	306	459	313	471
	251	378	257	386	263	395	269	404	274	412	280	421	286	429
	239	360	244	367	248	373	253	380	257	387	262	394	266	400
	224	336	227	341	230	346	233	351	237	356	240	361	243	366
W16×31	272	409	283	426	295	443	306	460	317	477	329	494	340	511
	260	390	269	405	279	420	289	435	299	449	309	464	319	479
	247	371	255	384	264	396	272	409	280	421	289	434	297	446
	234	352	241	362	248	372	254	382	261	393	268	403	275	413
	221	332	226	340	231	348	237	356	242	364	247	372	253	380
	208	313	212	319	216	325	220	331	224	337	229	343	233	350
	191	287	194	291	197	296	200	300	202	304	205	309	208	313

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W16–W14

$F_y = 50$ ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W16×26	110	166	TFL	0	384	189	284	198	298	208	312	217	327
			2	0.0863	337	184	276	192	289	201	302	209	314
			3	0.173	289	179	269	186	280	193	291	201	301
			4	0.259	242	174	261	180	270	186	279	192	288
			BFL	0.345	194	168	253	173	260	178	267	183	275
			6	2.04	145	161	241	164	247	168	252	171	258
			7	4.00	96.0	148	223	151	227	153	230	156	234
W14×38	153	231	TFL	0	558	252	379	266	400	280	421	294	442
			2	0.129	471	243	365	255	383	267	401	278	418
			3	0.258	384	234	351	243	365	253	380	262	394
			4	0.386	297	223	336	231	347	238	358	246	369
			BFL	0.515	209	213	320	218	328	223	335	228	343
			6	1.42	174	208	312	212	319	216	325	221	332
			7	2.55	140	201	302	204	307	208	312	211	318
W14×34	136	205	TFL	0	500	224	337	237	356	249	375	262	393
			2	0.114	423	216	325	227	341	238	357	248	373
			3	0.228	347	208	313	217	326	225	339	234	352
			4	0.341	270	199	300	206	310	213	320	220	330
			BFL	0.455	193	190	286	195	293	200	300	205	308
			6	1.41	159	185	279	189	285	193	291	197	297
			7	2.60	125	179	268	182	273	185	278	188	283
W14×30	118	177	TFL	0	442	197	296	208	313	219	329	230	346
			2	0.0963	378	190	286	200	300	209	314	219	329
			3	0.193	313	183	276	191	287	199	299	207	311
			4	0.289	248	176	265	182	274	189	283	195	293
			BFL	0.385	183	169	253	173	260	178	267	182	274
			6	1.48	147	163	246	167	251	171	257	174	262
			7	2.82	111	156	234	159	239	162	243	164	247
W14×26	100	151	TFL	0	385	172	258	181	273	191	287	201	302
			2	0.105	332	166	250	175	263	183	275	191	287
			3	0.210	279	161	242	168	252	175	263	182	273
			4	0.315	226	155	233	160	241	166	250	172	258
			BFL	0.420	174	149	223	153	230	157	236	162	243
			6	1.67	135	143	215	146	220	150	225	153	230
			7	3.18	96.1	134	202	137	206	139	209	142	213

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W16–W14

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
W16×26	**227**	341	**237**	356	**246**	370	**256**	385	**265**	399	**275**	413	**285**	428
	218	327	**226**	340	**234**	352	**243**	365	**251**	377	**260**	390	**268**	403
	208	312	**215**	323	**222**	334	**229**	345	**237**	356	**244**	366	**251**	377
	198	297	**204**	306	**210**	315	**216**	324	**222**	334	**228**	343	**234**	352
	188	282	**192**	289	**197**	296	**202**	304	**207**	311	**212**	318	**217**	326
	175	263	**179**	269	**182**	274	**186**	279	**190**	285	**193**	290	**197**	296
	158	237	**160**	241	**163**	245	**165**	248	**168**	252	**170**	255	**172**	259
W14×38	**308**	463	**322**	483	**336**	504	**350**	525	**363**	546	**377**	567	**391**	588
	290	436	**302**	454	**314**	471	**325**	489	**337**	507	**349**	524	**361**	542
	272	409	**281**	423	**291**	437	**301**	452	**310**	466	**320**	481	**329**	495
	253	380	**260**	391	**268**	403	**275**	414	**283**	425	**290**	436	**297**	447
	234	351	**239**	359	**244**	367	**249**	375	**255**	383	**260**	390	**265**	398
	225	338	**229**	345	**234**	351	**238**	358	**243**	365	**247**	371	**251**	378
	215	323	**218**	328	**222**	333	**225**	339	**229**	344	**232**	349	**236**	354
W14×34	**274**	412	**287**	431	**299**	450	**312**	468	**324**	487	**337**	506	**349**	525
	259	389	**269**	405	**280**	421	**290**	436	**301**	452	**311**	468	**322**	484
	243	365	**251**	378	**260**	391	**269**	404	**277**	417	**286**	430	**295**	443
	226	340	**233**	350	**240**	360	**247**	371	**253**	381	**260**	391	**267**	401
	209	315	**214**	322	**219**	329	**224**	337	**229**	344	**234**	351	**238**	358
	201	303	**205**	308	**209**	314	**213**	320	**217**	326	**221**	332	**225**	338
	191	287	**194**	292	**197**	297	**200**	301	**204**	306	**207**	311	**210**	315
W14×30	**241**	362	**252**	379	**263**	396	**274**	412	**285**	429	**296**	445	**307**	462
	228	343	**237**	357	**247**	371	**256**	385	**266**	399	**275**	413	**285**	428
	215	323	**222**	334	**230**	346	**238**	358	**246**	369	**254**	381	**261**	393
	201	302	**207**	311	**213**	321	**219**	330	**226**	339	**232**	348	**238**	358
	187	281	**191**	288	**196**	295	**201**	301	**205**	308	**210**	315	**214**	322
	178	268	**182**	273	**185**	279	**189**	284	**193**	290	**196**	295	**200**	301
	167	251	**170**	255	**173**	259	**175**	264	**178**	268	**181**	272	**184**	276
W14×26	**210**	316	**220**	330	**229**	345	**239**	359	**249**	374	**258**	388	**268**	402
	200	300	**208**	312	**216**	325	**224**	337	**233**	350	**241**	362	**249**	375
	189	283	**196**	294	**203**	304	**209**	315	**216**	325	**223**	336	**230**	346
	177	267	**183**	275	**189**	284	**194**	292	**200**	301	**206**	309	**211**	317
	166	249	**170**	256	**175**	262	**179**	269	**183**	275	**188**	282	**192**	288
	156	235	**160**	240	**163**	245	**166**	250	**170**	255	**173**	260	**177**	265
	144	216	**146**	220	**149**	224	**151**	227	**154**	231	**156**	234	**158**	238

ASD	**LRFD**
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W14–W12

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W14×22	**82.8**	125	TFL	0	325	**144**	216	**152**	228	**160**	240	**168**	253
			2	0.0838	283	**139**	210	**146**	220	**154**	231	**161**	241
			3	0.168	241	**135**	203	**141**	212	**147**	221	**153**	230
			4	0.251	199	**130**	196	**135**	203	**140**	211	**145**	218
			BFL	0.335	157	**126**	189	**130**	195	**133**	201	**137**	206
			6	1.69	119	**120**	181	**123**	185	**126**	189	**129**	194
			7	3.34	81.2	**112**	168	**114**	171	**116**	174	**118**	177
W12×30	**108**	162	TFL	0	440	**179**	269	**190**	286	**201**	302	**212**	319
			2	0.110	368	**172**	258	**181**	272	**190**	286	**199**	300
			3	0.220	296	**164**	247	**172**	258	**179**	269	**186**	280
			4	0.330	225	**156**	235	**162**	243	**167**	251	**173**	260
			BFL	0.440	153	**147**	222	**151**	227	**155**	233	**159**	239
			6	1.12	131	**145**	217	**148**	222	**151**	227	**154**	232
			7	1.94	110	**141**	212	**144**	216	**146**	220	**149**	224
W12×26	**92.8**	140	TFL	0	382	**155**	233	**164**	247	**174**	261	**183**	276
			2	0.0950	321	**148**	223	**156**	235	**164**	247	**172**	259
			3	0.190	259	**142**	213	**148**	223	**155**	233	**161**	242
			4	0.285	197	**135**	203	**140**	210	**145**	218	**150**	225
			BFL	0.380	136	**128**	192	**131**	197	**135**	202	**138**	207
			6	1.08	116	**125**	188	**128**	192	**131**	197	**134**	201
			7	1.95	95.6	**122**	183	**124**	186	**126**	190	**129**	194
W12×22	**73.1**	110	TFL	0	324	**132**	198	**140**	210	**148**	223	**156**	235
			2	0.106	281	**128**	192	**135**	202	**142**	213	**149**	223
			3	0.213	238	**123**	185	**129**	194	**135**	203	**141**	212
			4	0.319	196	**118**	177	**123**	185	**128**	192	**133**	199
			BFL	0.425	153	**113**	170	**117**	176	**121**	181	**124**	187
			6	1.66	117	**108**	162	**111**	166	**114**	171	**116**	175
			7	3.04	81.0	**99.9**	150	**102**	153	**104**	156	**106**	159
W12×19	**61.6**	92.6	TFL	0	279	**112**	169	**119**	179	**126**	190	**133**	200
			2	0.0875	244	**109**	164	**115**	173	**121**	182	**127**	191
			3	0.175	209	**105**	158	**110**	166	**115**	174	**121**	181
			4	0.263	174	**101**	152	**106**	159	**110**	165	**114**	172
			BFL	0.350	139	**97.2**	146	**101**	151	**104**	156	**108**	162
			6	1.65	104	**92.1**	138	**94.7**	142	**97.3**	146	**99.9**	150
			7	3.12	69.7	**84.6**	127	**86.3**	130	**88.1**	132	**89.8**	135

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W14–W12

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W14×22	176	265	184	277	192	289	200	301	209	313	217	326	225	338
	168	252	175	263	182	273	189	284	196	294	203	305	210	316
	159	239	165	248	171	257	177	266	183	275	189	284	195	293
	150	226	155	233	160	241	165	248	170	256	175	263	180	271
	141	212	145	218	149	224	153	230	157	236	161	242	165	248
	132	198	135	203	138	207	141	212	144	216	147	221	150	225
	120	180	122	183	124	186	126	189	128	192	130	195	132	198
W12×30	223	335	234	352	245	368	256	385	267	401	278	418	289	434
	209	314	218	327	227	341	236	355	245	369	255	383	264	396
	194	291	201	302	209	313	216	324	223	336	231	347	238	358
	178	268	184	277	190	285	195	294	201	302	206	310	212	319
	163	245	167	250	170	256	174	262	178	268	182	273	186	279
	158	237	161	242	164	247	167	252	171	257	174	262	177	266
	152	228	155	232	157	236	160	241	163	245	166	249	168	253
W12×26	193	290	202	304	212	319	221	333	231	347	241	362	250	376
	180	271	188	283	196	295	204	307	212	319	220	331	228	343
	168	252	174	262	181	271	187	281	194	291	200	301	206	310
	155	232	160	240	164	247	169	255	174	262	179	269	184	277
	141	212	145	218	148	223	151	228	155	233	158	238	162	243
	137	205	140	210	142	214	145	218	148	223	151	227	154	232
	131	197	134	201	136	204	138	208	141	211	143	215	145	219
W12×22	164	247	172	259	180	271	189	283	197	295	205	308	213	320
	156	234	163	244	170	255	177	265	184	276	191	287	198	297
	147	220	153	229	159	238	165	247	170	256	176	265	182	274
	138	207	142	214	147	221	152	229	157	236	162	244	167	251
	128	193	132	198	136	204	140	210	144	216	147	221	151	227
	119	179	122	184	125	188	128	193	131	197	134	201	137	206
	108	162	110	165	112	168	114	171	116	174	118	177	120	180
W12×19	140	211	147	221	154	232	161	242	168	252	175	263	182	273
	133	200	139	209	145	218	151	227	157	237	164	246	170	255
	126	189	131	197	136	205	142	213	147	221	152	228	157	236
	119	178	123	185	127	191	132	198	136	204	140	211	145	217
	111	167	114	172	118	177	121	182	125	188	128	193	132	198
	103	154	105	158	108	162	110	166	113	170	116	174	118	178
	91.5	138	93.3	140	95.0	143	96.8	145	98.5	148	100	151	102	153

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

W12–W10

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W12×16	**50.1**	75.4	TFL	0	236	**94.0**	141	**99.9**	150	**106**	159	**112**	168
			2	0.0663	209	**91.3**	137	**96.6**	145	**102**	153	**107**	161
			3	0.133	183	**88.6**	133	**93.1**	140	**97.7**	147	**102**	154
			4	0.199	156	**85.7**	129	**89.6**	135	**93.5**	141	**97.4**	146
			BFL	0.265	130	**82.8**	124	**86.0**	129	**89.3**	134	**92.5**	139
			6	1.70	94.4	**77.6**	117	**80.0**	120	**82.4**	124	**84.7**	127
			7	3.32	58.9	**69.6**	105	**71.1**	107	**72.6**	109	**74.0**	111
W12×14	**43.4**	65.3	TFL	0	208	**82.5**	124	**87.7**	132	**92.9**	140	**98.0**	147
			2	0.0563	185	**80.2**	121	**84.8**	128	**89.5**	134	**94.1**	141
			3	0.113	163	**77.9**	117	**82.0**	123	**86.0**	129	**90.1**	135
			4	0.169	141	**75.5**	114	**79.0**	119	**82.5**	124	**86.1**	129
			BFL	0.225	118	**73.1**	110	**76.0**	114	**79.0**	119	**81.9**	123
			6	1.69	85.2	**68.3**	103	**70.4**	106	**72.6**	109	**74.7**	112
			7	3.36	51.9	**60.8**	91.4	**62.1**	93.3	**63.4**	95.3	**64.7**	97.2
W10×26	**78.1**	117	TFL	0	381	**136**	205	**146**	219	**155**	233	**165**	247
			2	0.110	317	**130**	195	**137**	207	**145**	219	**153**	230
			3	0.220	254	**123**	184	**129**	194	**135**	203	**142**	213
			4	0.330	190	**115**	174	**120**	181	**125**	188	**130**	195
			BFL	0.440	127	**108**	162	**111**	167	**114**	172	**117**	177
			6	0.898	111	**106**	159	**109**	163	**111**	168	**114**	172
			7	1.51	95.1	**103**	155	**106**	159	**108**	163	**110**	166
W10×22	**64.9**	97.5	TFL	0	324	**115**	172	**123**	184	**131**	197	**139**	209
			2	0.0900	273	**109**	164	**116**	175	**123**	185	**130**	195
			3	0.180	221	**104**	156	**109**	164	**115**	173	**120**	181
			4	0.270	169	**98.1**	147	**102**	154	**107**	160	**111**	166
			BFL	0.360	117	**92.1**	138	**95.0**	143	**98.0**	147	**101**	152
			6	0.953	99.2	**89.8**	135	**92.3**	139	**94.7**	142	**97.2**	146
			7	1.71	81.1	**86.8**	130	**88.8**	133	**90.8**	137	**92.9**	140
W10×19	**53.9**	81.0	TFL	0	281	**99.8**	150	**107**	160	**114**	171	**121**	182
			2	0.0988	241	**95.7**	144	**102**	153	**108**	162	**114**	171
			3	0.198	201	**91.4**	137	**96.5**	145	**101**	153	**107**	160
			4	0.296	162	**87.0**	131	**91.0**	137	**95.1**	143	**99.1**	149
			BFL	0.395	122	**82.3**	124	**85.4**	128	**88.4**	133	**91.5**	137
			6	1.28	96.1	**78.8**	118	**81.2**	122	**83.6**	126	**86.0**	129
			7	2.31	70.2	**73.9**	111	**75.6**	114	**77.4**	116	**79.1**	119

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

W12–W10

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W12×16	118	177	123	185	129	194	135	203	141	212	147	221	153	230
	112	169	117	177	123	184	128	192	133	200	138	208	144	216
	107	161	111	167	116	174	120	181	125	188	130	195	134	202
	101	152	105	158	109	164	113	170	117	176	121	182	125	187
	95.7	144	99.0	149	102	154	105	159	109	163	112	168	115	173
	87.1	131	89.4	134	91.8	138	94.1	141	96.5	145	98.8	149	101	152
	75.5	113	77.0	116	78.4	118	79.9	120	81.4	122	82.8	125	84.3	127
W12×14	103	155	108	163	114	171	119	179	124	186	129	194	134	202
	98.7	148	103	155	108	162	113	169	117	176	122	183	126	190
	94.2	142	98.2	148	102	154	106	160	110	166	115	172	119	178
	89.6	135	93.1	140	96.6	145	100	150	104	156	107	161	111	166
	84.9	128	87.8	132	90.8	136	93.7	141	96.7	145	99.6	150	103	154
	76.8	115	79.0	119	81.1	122	83.2	125	85.3	128	87.5	131	89.6	135
	66.0	99.1	67.3	101	68.6	103	69.8	105	71.1	107	72.4	109	73.7	111
W10×26	174	262	184	276	193	290	203	304	212	319	222	333	231	347
	161	242	169	254	177	266	185	278	193	290	201	302	209	314
	148	223	154	232	161	242	167	251	173	261	180	270	186	280
	134	202	139	209	144	216	149	224	153	231	158	238	163	245
	121	181	124	186	127	191	130	196	133	200	136	205	140	210
	117	176	120	180	123	184	125	188	128	192	131	197	134	201
	113	170	115	173	118	177	120	180	122	184	125	187	127	191
W10×22	147	221	155	233	163	245	171	257	179	270	187	282	196	294
	137	205	143	215	150	226	157	236	164	246	171	256	177	267
	126	189	131	197	137	206	142	214	148	222	153	231	159	239
	115	173	119	179	123	185	128	192	132	198	136	204	140	211
	104	156	107	160	110	165	113	169	115	174	118	178	121	182
	99.7	150	102	154	105	157	107	161	110	165	112	168	115	172
	94.9	143	96.9	146	98.9	149	101	152	103	155	105	158	107	161
W10×19	128	192	135	203	142	213	149	224	156	234	163	245	170	255
	120	180	126	189	132	198	138	207	144	216	150	225	156	234
	112	168	117	175	122	183	127	190	132	198	137	205	142	213
	103	155	107	161	111	167	115	173	119	179	123	185	127	191
	94.5	142	97.6	147	101	151	104	156	107	160	110	165	113	170
	88.4	133	90.8	136	93.2	140	95.5	144	97.9	147	100	151	103	154
	80.9	122	82.6	124	84.4	127	86.2	129	87.9	132	89.7	135	91.4	137

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

W10

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

F_y = 50 ksi

Shape	M_p/Ω_b	$\phi_b M_p$	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.							
	kip-ft					2		2.5		3		3.5	
	ASD	LRFD		in.	kip	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W10×17	**46.7**	70.1	TFL	0	250	**87.9**	132	**94.1**	141	**100**	151	**107**	160
			2	0.0825	217	**84.5**	127	**89.9**	135	**95.3**	143	**101**	151
			3	0.165	183	**81.0**	122	**85.6**	129	**90.1**	135	**94.7**	142
			4	0.248	150	**77.3**	116	**81.1**	122	**84.8**	128	**88.6**	133
			BFL	0.330	117	**73.6**	111	**76.5**	115	**79.4**	119	**82.3**	124
			6	1.31	89.8	**69.8**	105	**72.1**	108	**74.3**	112	**76.6**	115
			7	2.46	62.4	**64.5**	96.9	**66.0**	99.3	**67.6**	102	**69.2**	104
W10×15	**39.9**	60.0	TFL	0	221	**77.0**	116	**82.5**	124	**88.0**	132	**93.5**	141
			2	0.0675	194	**74.3**	112	**79.1**	119	**84.0**	126	**88.8**	133
			3	0.135	167	**71.5**	107	**75.6**	114	**79.8**	120	**83.9**	126
			4	0.203	140	**68.6**	103	**72.0**	108	**75.5**	114	**79.0**	119
			BFL	0.270	113	**65.5**	98.5	**68.4**	103	**71.2**	107	**74.0**	111
			6	1.35	83.8	**61.6**	92.6	**63.7**	95.8	**65.8**	98.9	**67.9**	102
			7	2.60	55.1	**55.9**	84.1	**57.3**	86.1	**58.7**	88.2	**60.1**	90.3
W10×12	**31.4**	47.3	TFL	0	177	**61.2**	92.0	**65.6**	98.6	**70.0**	105	**74.4**	112
			2	0.0525	156	**59.1**	88.8	**63.0**	94.6	**66.9**	100	**70.8**	106
			3	0.105	135	**56.9**	85.6	**60.3**	90.6	**63.7**	95.7	**67.0**	101
			4	0.158	114	**54.7**	82.2	**57.6**	86.5	**60.4**	90.8	**63.3**	95.1
			BFL	0.210	93.6	**52.4**	78.8	**54.8**	82.3	**57.1**	85.8	**59.5**	89.4
			6	1.31	68.9	**49.2**	73.9	**50.9**	76.5	**52.6**	79.0	**54.3**	81.6
			7	2.61	44.2	**44.3**	66.5	**45.4**	68.2	**46.5**	69.9	**47.6**	71.5

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.

F_y = 50 ksi

Table 3–19 (continued)
Composite W Shapes
Available Strength in Flexure, kip-ft

W10

Shape	$Y2^b$, in.													
	4		4.5		5		5.5		6		6.5		7	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
W10×17	113	170	119	179	125	188	131	198	138	207	144	216	150	226
	106	159	112	168	117	176	122	184	128	192	133	200	139	208
	99.3	149	104	156	108	163	113	170	118	177	122	184	127	190
	92.3	139	96.1	144	99.8	150	104	156	107	161	111	167	115	173
	85.3	128	88.2	133	91.1	137	94.0	141	96.9	146	99.9	150	103	155
	78.8	118	81.0	122	83.3	125	85.5	129	87.8	132	90.0	135	92.2	139
	70.7	106	72.3	109	73.8	111	75.4	113	76.9	116	78.5	118	80.0	120
W10×15	99.0	149	105	157	110	165	116	174	121	182	127	190	132	198
	93.6	141	98.4	148	103	155	108	162	113	170	118	177	123	184
	88.1	132	92.3	139	96.4	145	101	151	105	157	109	164	113	170
	82.5	124	86.0	129	89.5	134	92.9	140	96.4	145	99.9	150	103	155
	76.8	115	79.6	120	82.4	124	85.2	128	88.0	132	90.8	137	93.7	141
	70.0	105	72.1	108	74.2	112	76.3	115	78.4	118	80.5	121	82.6	124
	61.4	92.3	62.8	94.4	64.2	96.5	65.6	98.5	66.9	101	68.3	103	69.7	105
W10×12	78.8	118	83.2	125	87.6	132	92.1	138	96.5	145	101	152	105	158
	74.6	112	78.5	118	82.4	124	86.3	130	90.2	136	94.1	141	98.0	147
	70.4	106	73.8	111	77.2	116	80.5	121	83.9	126	87.3	131	90.7	136
	66.1	99.4	69.0	104	71.8	108	74.7	112	77.5	117	80.4	121	83.2	125
	61.8	92.9	64.1	96.4	66.5	99.9	68.8	103	71.1	107	73.5	110	75.8	114
	56.0	84.2	57.7	86.8	59.5	89.4	61.2	92.0	62.9	94.5	64.6	97.1	66.3	99.7
	48.7	73.2	49.8	74.8	50.9	76.5	52.0	78.1	53.1	79.8	54.2	81.5	55.3	83.1

ASD	LRFD
$\Omega_b = 1.67$	$\phi_b = 0.90$

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

I_{LB}
W40

Table 3–20

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W40×297 (23200)	TFL	0	4370	44200	45200	46200	47200	48200	49300	50300	51400	52600	53700	54900
	2	0.413	3720	42500	43400	44300	45200	46200	47200	48200	49200	50200	51300	52300
	3	0.825	3060	40500	41300	42100	42900	43800	44700	45600	46500	47400	48300	49300
	4	1.24	2410	38100	38800	39500	40200	41000	41700	42500	43300	44100	44900	45700
	BFL	1.65	1760	35300	35800	36400	37000	37600	38200	38800	39400	40100	40700	41400
	6	4.59	1430	33600	34000	34500	35000	35500	36100	36600	37100	37700	38200	38800
	7	8.17	1090	31600	32000	32400	32800	33200	33600	34000	34500	34900	35400	35800
W40×294 (21900)	TFL	0	4310	43100	44100	45100	46100	47100	48200	49300	50400	51500	52600	53800
	2	0.483	3730	41600	42500	43400	44400	45300	46300	47300	48300	49400	50400	51500
	3	0.965	3150	39800	40700	41500	42300	43200	44100	45000	45900	46900	47800	48800
	4	1.45	2580	37800	38500	39300	40000	40800	41600	42400	43200	44100	44900	45800
	BFL	1.93	2000	35400	36000	36600	37200	37900	38600	39200	39900	40700	41400	42100
	6	5.69	1540	33100	33600	34100	34600	35200	35700	36300	36900	37500	38100	38700
	7	10.00	1080	30400	30800	31200	31600	32000	32400	32900	33300	33800	34200	34700
W40×278 (20500)	TFL	0	4100	40500	41400	42300	43300	44300	45300	46300	47300	48400	49500	50600
	2	0.453	3560	39100	39900	40800	41700	42600	43500	44500	45400	46400	47400	48400
	3	0.905	3020	37500	38200	39000	39800	40700	41500	42400	43300	44200	45100	46000
	4	1.36	2470	35500	36200	36900	37600	38400	39100	39900	40700	41500	42300	43100
	BFL	1.81	1930	33300	33900	34500	35100	35700	36400	37000	37700	38400	39000	39700
	6	5.65	1480	31100	31600	32100	32600	33100	33600	34200	34700	35300	35900	36400
	7	10.1	1020	28500	28800	29200	29600	30000	30400	30800	31200	31600	32000	32500
W40×277 (21900)	TFL	0	4070	41300	42200	43100	44100	45000	46000	47000	48000	49100	50100	51200
	2	0.394	3450	39700	40500	41400	42300	43100	44000	45000	45900	46800	47800	48800
	3	0.788	2820	37800	38500	39300	40100	40800	41600	42500	43300	44200	45000	45900
	4	1.18	2200	35500	36200	36800	37500	38100	38800	39500	40200	41000	41700	42500
	BFL	1.58	1580	32800	33300	33800	34300	34800	35400	36000	36500	37100	37700	38300
	6	4.22	1300	31300	31700	32200	32600	33100	33600	34100	34600	35100	35600	36100
	7	7.59	1020	29700	30000	30400	30800	31200	31600	32000	32400	32800	33200	33700
W40×264 (19400)	TFL	0	3880	38200	39000	39900	40800	41700	42700	43600	44600	45600	46600	47700
	2	0.433	3360	36800	37600	38400	39300	40100	41000	41900	42800	43700	44700	45600
	3	0.865	2850	35300	36000	36800	37500	38300	39100	39900	40800	41600	42500	43400
	4	1.30	2330	33500	34100	34800	35500	36200	36900	37600	38300	39100	39800	40600
	BFL	1.73	1810	31300	31900	32500	33000	33600	34200	34800	35400	36100	36700	37400
	6	5.50	1390	29300	29800	30200	30700	31200	31700	32200	32700	33200	33800	34300
	7	9.90	969	26900	27200	27600	28000	28300	28700	29100	29500	29900	30300	30700

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 3–20 (continued)

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

I_{LB}

W40

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W40×249 (19600)	TFL	0	3670	36900	37700	38500	39300	40200	41100	42000	42900	43800	44800	45700
	2	0.355	3110	35400	36200	36900	37700	38500	39300	40100	41000	41800	42700	43600
	3	0.710	2550	33800	34400	35100	35800	36500	37200	37900	38700	39500	40200	41000
	4	1.07	1990	31700	32300	32900	33500	34100	34700	35300	36000	36600	37300	38000
	BFL	1.42	1430	29300	29700	30200	30700	31100	31600	32100	32700	33200	33700	34300
	6	4.04	1170	27900	28300	28700	29100	29600	30000	30400	30900	31300	31800	32200
	7	7.47	917	26500	26800	27200	27500	27800	28200	28500	28900	29300	29700	30100
W40×235 (17400)	TFL	0	3450	33900	34600	35400	36200	37000	37800	38700	39600	40400	41300	42300
	2	0.394	2980	32700	33400	34100	34800	35600	36400	37100	37900	38800	39600	40400
	3	0.788	2510	31300	31900	32600	33200	33900	34600	35300	36100	36800	37600	38300
	4	1.18	2040	29600	30200	30800	31400	32000	32600	33200	33900	34500	35200	35900
	BFL	1.58	1580	27700	28200	28700	29200	29700	30200	30800	31300	31900	32400	33000
	6	5.16	1220	26000	26400	26800	27200	27600	28100	28500	29000	29400	29900	30400
	7	9.46	862	24000	24300	24600	24900	25200	25600	25900	26300	26600	27000	27300
W40×215 (16700)	TFL	0	3170	31300	32000	32700	33500	34200	34900	35700	36500	37300	38100	38900
	2	0.305	2690	30100	30800	31400	32100	32800	33400	34200	34900	35600	36400	37100
	3	0.610	2210	28700	29300	29900	30500	31100	31700	32300	33000	33600	34300	35000
	4	0.915	1730	27000	27500	28000	28500	29000	29600	30100	30700	31200	31800	32400
	BFL	1.22	1250	25000	25400	25800	26200	26600	27000	27500	27900	28300	28800	29300
	6	3.80	1020	23800	24200	24500	24900	25200	25600	26000	26300	26700	27100	27500
	7	7.30	792	22600	22800	23100	23400	23700	24000	24300	24600	24900	25300	25600
W40×211 (15500)	TFL	0	3100	30100	30800	31500	32200	32900	33600	34400	35200	36000	36800	37600
	2	0.354	2680	29000	29700	30300	31000	31600	32300	33000	33700	34500	35200	36000
	3	0.708	2270	27800	28400	29000	29600	30200	30800	31500	32100	32800	33500	34200
	4	1.06	1850	26400	26900	27400	28000	28500	29100	29600	30200	30800	31400	32000
	BFL	1.42	1430	24700	25100	25600	26000	26500	26900	27400	27900	28400	28900	29400
	6	4.98	1100	23100	23500	23900	24200	24600	25000	25400	25800	26200	26600	27100
	7	9.35	775	21300	21600	21900	22200	22500	22800	23100	23400	23700	24000	24300
W40×199 (14900)	TFL	0	2920	28200	28800	29500	30100	30800	31500	32200	32900	33700	34400	35200
	2	0.266	2510	27200	27800	28400	29000	29600	30200	30900	31600	32200	32900	33600
	3	0.533	2090	26000	26500	27100	27600	28200	28700	29300	29900	30500	31200	31800
	4	0.799	1670	24600	25000	25500	26000	26500	27000	27500	28000	28500	29100	29600
	BFL	1.07	1250	22900	23300	23600	24000	24400	24900	25300	25700	26100	26600	27000
	6	4.11	989	21600	21900	22300	22600	22900	23300	23700	24000	24400	24800	25200
	7	8.09	731	20200	20500	20700	21000	21300	21500	21800	22100	22400	22700	23000

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

I_{LB}
W40–W36

Table 3–20 (continued)
Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W40×183 (13200)	TFL	0	2670	25500	26100	26700	27300	27900	28600	29200	29900	30500	31200	31900
	2	0.300	2310	24600	25200	25700	26300	26900	27400	28000	28700	29300	29900	30600
	3	0.600	1960	23600	24100	24600	25100	25700	26200	26700	27300	27900	28500	29100
	4	0.900	1600	22400	22900	23300	23800	24200	24700	25200	25700	26200	26700	27200
	BFL	1.20	1250	21100	21400	21800	22200	22600	23000	23400	23800	24300	24700	25100
	6	4.76	958	19700	20000	20300	20600	21000	21300	21700	22000	22400	22700	23100
	7	9.24	666	18100	18400	18600	18800	19100	19300	19600	19900	20100	20400	20700
W40×167 (11600)	TFL	0	2460	22800	23300	23800	24400	24900	25500	26100	26700	27300	28000	28600
	2	0.256	2160	22000	22500	23000	23500	24100	24600	25200	25700	26300	26900	27500
	3	0.513	1850	21200	21600	22100	22600	23100	23600	24100	24600	25100	25600	26200
	4	0.769	1550	20200	20600	21000	21500	21900	22400	22800	23300	23800	24200	24700
	BFL	1.03	1250	19100	19500	19800	20200	20600	21000	21400	21800	22200	22600	23100
	6	4.97	931	17700	18000	18300	18600	18900	19200	19600	19900	20200	20600	20900
	7	9.84	614	16100	16300	16500	16700	16900	17200	17400	17600	17900	18100	18400
W40×149 (9800)	TFL	0	2190	19600	20000	20500	21000	21500	22000	22500	23100	23600	24200	24700
	2	0.208	1950	19000	19400	19900	20300	20800	21300	21800	22300	22800	23300	23900
	3	0.415	1700	18300	18700	19200	19600	20000	20500	20900	21400	21900	22300	22800
	4	0.623	1460	17600	18000	18400	18800	19200	19600	20000	20400	20800	21300	21700
	BFL	0.830	1210	16700	17100	17400	17800	18100	18500	18900	19200	19600	20000	20400
	6	5.14	879	15400	15700	15900	16200	16500	16800	17100	17400	17700	18000	18300
	7	10.4	548	13700	13900	14100	14300	14500	14700	14900	15100	15300	15500	15800
W36×302 (21100)	TFL	0	4440	40100	41000	41900	42900	43900	44900	46000	47000	48100	49200	50400
	2	0.420	3740	38400	39300	40200	41100	42000	42900	43800	44800	45800	46800	47800
	3	0.840	3040	36500	37300	38000	38800	39600	40500	41300	42200	43100	44000	44900
	4	1.26	2340	34200	34800	35500	36200	36800	37500	38300	39000	39700	40500	41300
	BFL	1.68	1640	31300	31800	32300	32900	33400	34000	34500	35100	35700	36300	36900
	6	4.09	1380	30100	30500	31000	31400	31900	32400	32900	33400	33900	34400	35000
	7	6.91	1110	28700	29100	29400	29800	30200	30600	31000	31500	31900	32300	32800
W36×282 (19600)	TFL	0	4150	37100	38000	38900	39800	40700	41700	42600	43600	44600	45600	46700
	2	0.393	3500	35600	36400	37200	38100	38900	39800	40700	41600	42500	43400	44400
	3	0.785	2840	33800	34500	35300	36000	36800	37500	38300	39100	39900	40800	41600
	4	1.18	2190	31700	32300	32900	33500	34200	34800	35500	36200	36900	37600	38300
	BFL	1.57	1540	29100	29600	30000	30500	31000	31500	32100	32600	33100	33700	34300
	6	3.99	1290	27900	28300	28700	29200	29600	30100	30500	31000	31500	32000	32500
	7	6.84	1040	26600	27000	27300	27700	28100	28400	28800	29200	29600	30000	30500

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 3–20 (continued)

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

I_{LB}

W36

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×262	TFL	0	3850	34000	34800	35600	36400	37300	38100	39000	39900	40900	41800	42800
(17900)	2	0.360	3250	32600	33300	34100	34800	35600	36400	37200	38100	38900	39800	40700
	3	0.720	2660	31000	31700	32300	33000	33700	34400	35200	35900	36700	37400	38200
	4	1.08	2060	29100	29700	30200	30800	31400	32000	32600	33300	33900	34600	35200
	BFL	1.44	1470	26800	27200	27700	28100	28600	29100	29600	30100	30600	31100	31700
	6	3.98	1210	25600	26000	26400	26800	27200	27600	28000	28400	28900	29300	29800
	7	6.97	962	24300	24600	25000	25300	25600	26000	26300	26700	27100	27500	27900
W36×256	TFL	0	3770	33000	33800	34600	35400	36200	37100	38000	38900	39800	40800	41700
(16800)	2	0.433	3240	31800	32500	33200	34000	34800	35600	36400	37200	38100	39000	39800
	3	0.865	2710	30300	31000	31700	32400	33100	33800	34500	35300	36100	36800	37600
	4	1.30	2180	28700	29200	29800	30400	31100	31700	32300	33000	33700	34400	35100
	BFL	1.73	1650	26600	27100	27600	28100	28600	29200	29700	30300	30800	31400	32000
	6	5.19	1300	25100	25500	25900	26300	26800	27200	27700	28100	28600	29100	29600
	7	8.90	942	23300	23600	23900	24200	24600	24900	25300	25600	26000	26400	26800
W36×247	TFL	0	3630	31700	32400	33200	34000	34800	35600	36500	37300	38200	39100	40000
(16700)	2	0.338	3070	30400	31100	31800	32600	33300	34000	34800	35600	36400	37200	38000
	3	0.675	2510	29000	29600	30200	30800	31500	32200	32900	33600	34300	35000	35700
	4	1.01	1950	27200	27700	28200	28800	29400	29900	30500	31100	31700	32300	33000
	BFL	1.35	1400	25100	25500	25900	26300	26800	27200	27700	28200	28700	29200	29700
	6	3.93	1150	23900	24300	24600	25000	25400	25800	26200	26600	27000	27500	27900
	7	7.00	907	22700	23000	23300	23600	23900	24300	24600	24900	25300	25600	26000
W36×232	TFL	0	3410	29400	30100	30800	31600	32300	33100	33900	34700	35600	36400	37300
(15000)	2	0.393	2930	28300	29000	29600	30300	31000	31700	32500	33200	34000	34800	35600
	3	0.785	2450	27000	27600	28200	28900	29500	30200	30800	31500	32200	32900	33600
	4	1.18	1980	25600	26100	26600	27200	27700	28300	28900	29500	30100	30700	31400
	BFL	1.57	1500	23800	24200	24700	25100	25600	26100	26600	27100	27600	28100	28600
	6	5.03	1180	22400	22800	23100	23500	23900	24300	24700	25100	25600	26000	26500
	7	8.77	851	20800	21000	21300	21600	21900	22200	22600	22900	23200	23600	23900
W36×231	TFL	0	3400	29500	30200	31000	31700	32400	33200	34000	34800	35600	36400	37300
(15600)	2	0.315	2890	28400	29100	29700	30400	31100	31800	32500	33200	34000	34700	35500
	3	0.630	2370	27100	27600	28200	28800	29400	30100	30700	31400	32000	32700	33400
	4	0.95	1850	25400	25900	26400	26900	27500	28000	28600	29100	29700	30300	30900
	BFL	1.26	1330	23400	23800	24200	24600	25100	25500	25900	26400	26800	27300	27800
	6	3.90	1090	22400	22700	23100	23400	23800	24100	24500	24900	25300	25700	26100
	7	7.05	851	21200	21500	21700	22000	22300	22600	23000	23300	23600	23900	24300

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

I_{LB}
W36

Table 3–20 (continued)
Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×210	TFL	0	3090	26000	26600	27300	27900	28600	29300	30000	30800	31500	32300	33100
(13200)	2	0.340	2680	25100	25700	26300	26900	27500	28200	28800	29500	30200	30900	31600
	3	0.680	2260	24000	24500	25100	25700	26200	26800	27400	28000	28700	29300	30000
	4	1.02	1850	22800	23300	23700	24200	24800	25300	25800	26400	26900	27500	28100
	BFL	1.36	1430	21300	21700	22100	22500	23000	23400	23900	24300	24800	25300	25800
	6	5.05	1100	19900	20300	20600	20900	21300	21700	22000	22400	22800	23200	23600
	7	9.04	773	18300	18600	18800	19100	19400	19700	19900	20200	20500	20800	21100
W36×194	TFL	0	2850	23800	24400	25000	25600	26200	26800	27500	28200	28900	29600	30300
(12100)	2	0.315	2470	22900	23500	24000	24600	25200	25800	26400	27000	27700	28300	29000
	3	0.630	2090	22000	22500	23000	23500	24000	24600	25100	25700	26300	26900	27500
	4	0.945	1710	20900	21300	21700	22200	22700	23200	23600	24200	24700	25200	25700
	BFL	1.26	1320	19500	19900	20200	20600	21000	21400	21800	22300	22700	23100	23600
	6	4.94	1020	18300	18600	18900	19200	19500	19900	20200	20600	20900	21300	21700
	7	8.93	713	16800	17000	17200	17500	17700	18000	18300	18500	18800	19100	19400
W36×182	TFL	0	2680	22200	22700	23300	23900	24500	25100	25700	26300	27000	27600	28300
(11300)	2	0.295	2320	21400	21900	22400	23000	23500	24100	24600	25200	25800	26400	27100
	3	0.590	1970	20500	21000	21500	22000	22500	23000	23500	24000	24600	25100	25700
	4	0.885	1610	19500	19900	20300	20700	21200	21600	22100	22600	23000	23500	24000
	BFL	1.18	1260	18300	18600	19000	19300	19700	20100	20500	20900	21300	21700	22200
	6	4.88	963	17100	17400	17600	18000	18300	18600	18900	19200	19600	19900	20300
	7	8.92	670	15700	15900	16100	16300	16600	16800	17100	17300	17600	17800	18100
W36×170	TFL	0	2500	20600	21100	21600	22200	22700	23300	23800	24400	25000	25600	26200
(10500)	2	0.275	2170	19900	20300	20800	21300	21800	22400	22900	23400	24000	24500	25100
	3	0.550	1840	19100	19500	19900	20400	20800	21300	21800	22300	22800	23300	23800
	4	0.825	1510	18100	18500	18900	19300	19700	20100	20500	21000	21400	21900	22400
	BFL	1.10	1180	17000	17300	17600	18000	18300	18700	19000	19400	19800	20200	20600
	6	4.81	902	15800	16100	16400	16700	17000	17300	17600	17900	18200	18500	18800
	7	8.88	626	14500	14700	15000	15200	15400	15600	15800	16100	16300	16600	16800
W36×160	TFL	0	2350	19200	19600	20100	20600	21100	21700	22200	22700	23300	23900	24500
(9760)	2	0.255	2050	18500	19000	19400	19900	20400	20900	21400	21900	22400	22900	23500
	3	0.510	1740	17800	18200	18600	19000	19400	19900	20300	20800	21300	21800	22300
	4	0.765	1430	16900	17200	17600	18000	18400	18800	19200	19600	20000	20400	20900
	BFL	1.02	1130	15900	16200	16500	16800	17200	17500	17800	18200	18600	18900	19300
	6	4.80	858	14800	15000	15300	15600	15800	16100	16400	16700	17000	17300	17600
	7	8.96	588	13500	13700	13900	14100	14300	14500	14700	15000	15200	15400	15600

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 3–20 (continued)
Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

W36–W33

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W36×150 (9040)	TFL	0	2210	17800	18300	18700	19200	19700	20200	20700	21200	21700	22200	22800
	2	0.235	1930	17200	17600	18100	18500	18900	19400	19900	20400	20800	21300	21800
	3	0.470	1650	16500	16900	17300	17700	18100	18500	19000	19400	19900	20300	20800
	4	0.705	1370	15800	16100	16400	16800	17200	17500	17900	18300	18700	19100	19600
	BFL	0.940	1090	14800	15100	15400	15700	16100	16400	16700	17100	17400	17800	18100
	6	4.82	819	13800	14000	14300	14500	14800	15100	15300	15600	15900	16200	16500
	7	9.08	553	12600	12700	12900	13100	13300	13500	13700	13900	14100	14300	14500
W36×135 (7800)	TFL	0	1990	15600	16000	16400	16800	17200	17700	18100	18600	19000	19500	20000
	2	0.198	1750	15100	15500	15800	16200	16600	17000	17500	17900	18300	18800	19200
	3	0.395	1520	14500	14900	15200	15600	16000	16300	16700	17100	17500	18000	18400
	4	0.593	1280	13900	14200	14500	14800	15200	15500	15900	16200	16600	17000	17400
	BFL	0.790	1040	13100	13400	13700	14000	14300	14600	14900	15200	15500	15800	16200
	6	4.94	770	12100	12400	12600	12800	13100	13300	13600	13800	14100	14300	14600
	7	9.49	497	10900	11100	11200	11400	11600	11700	11900	12100	12300	12500	12700
W33×221 (12900)	TFL	0	3260	24600	25200	25900	26500	27200	27900	28600	29300	30100	30800	31600
	2	0.319	2750	23600	24200	24800	25400	26000	26600	27300	28000	28600	29300	30000
	3	0.638	2250	22500	23000	23500	24000	24600	25200	25700	26300	26900	27600	28200
	4	0.956	1750	21100	21500	22000	22400	22900	23400	23900	24400	24900	25400	26000
	BFL	1.28	1240	19400	19700	20100	20400	20800	21200	21600	22000	22400	22800	23200
	6	3.69	1030	18500	18800	19100	19500	19800	20100	20500	20800	21200	21500	21900
	7	6.46	814	17600	17800	18100	18400	18600	18900	19200	19500	19800	20100	20400
W33×201 (11600)	TFL	0	2960	22100	22700	23200	23800	24500	25100	25700	26400	27000	27700	28400
	2	0.288	2510	21200	21800	22300	22800	23400	24000	24600	25200	25800	26400	27000
	3	0.575	2050	20200	20700	21100	21600	22100	22600	23200	23700	24200	24800	25400
	4	0.863	1600	19000	19400	19800	20200	20600	21100	21500	22000	22400	22900	23400
	BFL	1.15	1150	17500	17800	18100	18500	18800	19100	19500	19900	20200	20600	21000
	6	3.64	944	16700	17000	17200	17500	17800	18100	18400	18700	19100	19400	19700
	7	6.49	740	15800	16000	16300	16500	16700	17000	17200	17500	17800	18000	18300
W33×169 (9290)	TFL	0	2480	18100	18600	19100	19600	20100	20600	21200	21700	22300	22900	23500
	2	0.305	2120	17500	17900	18300	18800	19300	19800	20300	20800	21300	21800	22300
	3	0.610	1770	16700	17100	17500	17900	18300	18800	19200	19700	20100	20600	21100
	4	0.915	1420	15700	16100	16400	16800	17200	17600	18000	18400	18800	19200	19600
	BFL	1.22	1070	14600	14900	15200	15500	15800	16100	16500	16800	17100	17500	17800
	6	4.29	846	13800	14000	14300	14500	14800	15100	15300	15600	15900	16200	16500
	7	7.67	619	12800	13000	13200	13400	13600	13800	14000	14300	14500	14700	15000

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

I_{LB}

W33–W30

Table 3–20 (continued)

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W33×152	TFL	0	2240	16000	16500	16900	17300	17800	18300	18700	19200	19700	20300	20800
(8160)	2	0.264	1930	15400	15800	16200	16700	17100	17500	18000	18400	18900	19400	19800
	3	0.528	1630	14800	15100	15500	15900	16300	16700	17100	17500	17900	18400	18800
	4	0.791	1320	14000	14300	14600	15000	15300	15700	16000	16400	16800	17100	17500
	BFL	1.06	1020	13100	13300	13600	13900	14200	14500	14800	15100	15400	15700	16100
	6	4.33	788	12300	12500	12700	12900	13200	13400	13700	13900	14200	14500	14700
	7	7.94	559	11300	11500	11600	11800	12000	12200	12400	12600	12800	13000	13200
W33×141	TFL	0	2080	14700	15100	15500	15900	16300	16700	17200	17600	18100	18600	19100
(7450)	2	0.240	1800	14200	14500	14900	15300	15700	16100	16500	16900	17300	17800	18200
	3	0.480	1520	13600	13900	14200	14600	14900	15300	15700	16100	16500	16900	17300
	4	0.720	1250	12900	13200	13500	13800	14100	14400	14800	15100	15500	15800	16200
	BFL	0.960	970	12100	12300	12600	12800	13100	13400	13600	13900	14200	14500	14800
	6	4.34	745	11300	11500	11700	11900	12100	12400	12600	12800	13100	13300	13600
	7	8.06	519	10300	10500	10700	10800	11000	11200	11300	11500	11700	11900	12100
W33×130	TFL	0	1920	13300	13700	14000	14400	14800	15200	15600	16000	16500	16900	17300
(6710)	2	0.214	1670	12800	13200	13500	13900	14200	14600	15000	15400	15800	16200	16600
	3	0.428	1420	12300	12600	12900	13300	13600	13900	14300	14600	15000	15400	15800
	4	0.641	1180	11700	12000	12300	12600	12900	13200	13500	13800	14100	14500	14800
	BFL	0.855	931	11000	11300	11500	11700	12000	12300	12500	12800	13100	13400	13700
	6	4.39	705	10300	10400	10600	10900	11100	11300	11500	11700	11900	12200	12400
	7	8.29	479	9350	9490	9640	9790	9940	10100	10300	10400	10600	10800	11000
W33×118	TFL	0	1730	11800	12100	12400	12800	13100	13500	13900	14200	14600	15000	15400
(5900)	2	0.185	1520	11400	11700	12000	12300	12700	13000	13300	13700	14000	14400	14800
	3	0.370	1310	11000	11200	11500	11800	12100	12400	12800	13100	13400	13700	14100
	4	0.555	1100	10500	10700	11000	11200	11500	11800	12100	12400	12700	13000	13300
	BFL	0.740	884	9880	10100	10300	10600	10800	11000	11300	11500	11800	12100	12300
	6	4.45	659	9140	9320	9510	9690	9890	10100	10300	10500	10700	10900	11100
	7	8.55	433	8250	8380	8520	8650	8790	8940	9080	9230	9390	9540	9700
W30×116	TFL	0	1710	9870	10200	10500	10800	11100	11400	11800	12100	12500	12800	13200
(4930)	2	0.213	1490	9530	9810	10100	10400	10700	11000	11300	11600	12000	12300	12600
	3	0.425	1260	9130	9380	9630	9900	10200	10400	10700	11000	11300	11600	12000
	4	0.638	1040	8670	8890	9120	9360	9600	9850	10100	10400	10600	10900	11200
	BFL	0.850	818	8130	8320	8520	8720	8930	9140	9360	9580	9810	10000	10300
	6	3.98	623	7570	7730	7890	8060	8230	8400	8580	8770	8960	9150	9350
	7	7.44	427	6910	7020	7150	7270	7400	7530	7660	7800	7950	8090	8240

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 3–20 (continued)

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

I_{LB}

W30–W27

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W30×108	TFL	0	1590	9010	9290	9570	9850	10200	10500	10800	11100	11400	11800	12100
(4470)	2	0.190	1390	8710	8960	9220	9490	9770	10100	10300	10600	11000	11300	11600
	3	0.380	1190	8360	8590	8830	9080	9330	9600	9860	10100	10400	10700	11000
	4	0.570	989	7960	8160	8380	8600	8830	9060	9300	9550	9800	10100	10300
	BFL	0.760	790	7490	7670	7850	8050	8240	8450	8650	8870	9080	9310	9530
	6	4.03	593	6940	7090	7240	7400	7560	7720	7890	8070	8250	8430	8620
	7	7.64	396	6280	6390	6500	6620	6740	6860	6980	7110	7240	7380	7510
W30×99	TFL	0	1450	8100	8350	8600	8870	9140	9410	9700	9990	10300	10600	10900
(3990)	2	0.168	1280	7840	8080	8320	8560	8820	9080	9340	9610	9900	10200	10500
	3	0.335	1100	7540	7750	7970	8200	8430	8670	8910	9160	9420	9690	9960
	4	0.503	929	7200	7390	7590	7800	8010	8220	8450	8670	8910	9150	9390
	BFL	0.670	754	6800	6970	7150	7320	7510	7700	7890	8090	8300	8510	8720
	6	4.08	559	6280	6410	6560	6700	6850	7010	7160	7330	7490	7660	7840
	7	7.83	364	5640	5740	5840	5950	6050	6160	6280	6400	6520	6640	6760
W30×90	TFL	0	1320	7320	7540	7770	8010	8250	8510	8760	9030	9300	9570	9860
(3610)	2	0.153	1160	7080	7290	7500	7730	7950	8190	8430	8680	8930	9190	9460
	3	0.305	1000	6810	7000	7200	7400	7610	7830	8050	8280	8510	8750	9000
	4	0.458	842	6500	6670	6850	7040	7230	7420	7620	7830	8040	8260	8480
	BFL	0.610	683	6140	6290	6450	6610	6780	6950	7120	7300	7490	7680	7870
	6	3.99	506	5670	5790	5920	6050	6180	6320	6470	6610	6760	6920	7070
	7	7.76	329	5090	5180	5270	5370	5460	5560	5670	5770	5880	5990	6100
W27×102	TFL	0	1500	7250	7480	7730	7980	8240	8510	8780	9060	9350	9650	9950
(3620)	2	0.208	1290	6970	7190	7420	7650	7890	8140	8390	8650	8920	9200	9480
	3	0.415	1090	6670	6870	7080	7290	7510	7730	7960	8200	8440	8690	8950
	4	0.623	879	6300	6470	6660	6840	7030	7230	7430	7640	7860	8080	8300
	BFL	0.830	671	5860	6010	6160	6310	6480	6640	6810	6980	7160	7350	7530
	6	3.38	523	5490	5620	5740	5870	6010	6150	6290	6430	6580	6740	6890
	7	6.26	375	5070	5160	5260	5360	5470	5570	5680	5800	5910	6030	6150
W27×94	TFL	0	1380	6570	6790	7010	7240	7480	7730	7980	8240	8500	8770	9050
(3270)	2	0.186	1200	6340	6540	6750	6970	7190	7410	7650	7890	8140	8390	8650
	3	0.373	1010	6060	6240	6430	6630	6830	7040	7250	7470	7690	7920	8160
	4	0.559	824	5740	5900	6070	6240	6420	6600	6790	6980	7180	7380	7590
	BFL	0.745	638	5360	5490	5640	5780	5930	6090	6250	6410	6580	6750	6920
	6	3.43	492	5000	5120	5240	5360	5480	5610	5740	5880	6020	6160	6310
	7	6.41	346	4590	4680	4770	4860	4960	5060	5160	5260	5370	5480	5590

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

LB
W27–W24

Table 3–20 (continued)
Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W27×84	TFL	0	1240	5770	5960	6160	6370	6580	6800	7020	7250	7490	7730	7980
(2850)	2	0.160	1080	5570	5750	5940	6130	6320	6530	6740	6950	7170	7400	7630
	3	0.320	918	5340	5500	5670	5840	6030	6210	6400	6600	6800	7010	7220
	4	0.480	758	5070	5210	5360	5520	5680	5850	6020	6190	6370	6560	6750
	BFL	0.640	598	4750	4880	5010	5140	5280	5420	5570	5720	5870	6030	6190
	6	3.50	454	4420	4520	4630	4740	4850	4970	5090	5210	5340	5470	5600
	7	6.63	309	4020	4090	4170	4250	4340	4430	4520	4610	4700	4800	4900
W24×94	TFL	0	1380	5470	5670	5880	6090	6310	6530	6770	7010	7260	7510	7770
(2700)	2	0.219	1190	5260	5450	5640	5840	6040	6250	6470	6690	6920	7150	7400
	3	0.438	988	5010	5180	5350	5530	5710	5900	6090	6290	6500	6710	6930
	4	0.656	790	4720	4860	5010	5170	5330	5490	5660	5840	6020	6200	6390
	BFL	0.875	592	4360	4480	4600	4730	4860	5000	5140	5290	5430	5590	5740
	6	3.05	469	4100	4200	4310	4420	4530	4640	4760	4880	5010	5140	5270
	7	5.43	346	3810	3890	3970	4060	4140	4240	4330	4430	4520	4630	4730
W24×84	TFL	0	1240	4810	4990	5170	5360	5560	5760	5970	6180	6400	6630	6860
(2370)	2	0.193	1060	4620	4790	4950	5130	5310	5500	5690	5880	6090	6300	6510
	3	0.385	888	4410	4560	4710	4870	5030	5200	5370	5550	5740	5930	6120
	4	0.578	715	4160	4290	4420	4560	4700	4850	5000	5160	5320	5490	5660
	BFL	0.770	541	3860	3960	4070	4190	4310	4430	4560	4690	4820	4960	5100
	6	3.01	425	3620	3710	3800	3900	4000	4100	4210	4320	4430	4550	4670
	7	5.48	309	3350	3420	3490	3570	3640	3720	3810	3890	3980	4070	4160
W24×76	TFL	0	1120	4280	4440	4600	4770	4950	5130	5320	5510	5710	5910	6120
(2100)	2	0.170	966	4120	4270	4420	4580	4740	4910	5080	5260	5440	5630	5830
	3	0.340	813	3930	4070	4210	4350	4500	4650	4810	4970	5140	5310	5480
	4	0.510	660	3720	3840	3960	4080	4210	4350	4490	4630	4780	4930	5080
	BFL	0.680	507	3460	3560	3660	3770	3880	3990	4110	4230	4350	4480	4610
	6	3.02	393	3230	3320	3400	3490	3580	3680	3770	3870	3980	4080	4190
	7	5.61	280	2970	3040	3100	3170	3240	3310	3390	3470	3540	3630	3710
W24×68	TFL	0	1000	3760	3900	4040	4190	4350	4510	4680	4850	5030	5210	5390
(1830)	2	0.146	872	3620	3760	3890	4030	4180	4330	4480	4640	4810	4980	5150
	3	0.293	741	3470	3590	3710	3840	3980	4110	4250	4400	4550	4710	4860
	4	0.439	610	3290	3400	3510	3620	3740	3860	3990	4120	4250	4390	4530
	BFL	0.585	479	3080	3170	3260	3360	3460	3570	3670	3790	3900	4020	4140
	6	3.07	365	2860	2930	3010	3090	3180	3260	3350	3440	3540	3640	3730
	7	5.82	251	2600	2660	2720	2780	2840	2910	2970	3040	3110	3180	3260

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 3–20 (continued)
Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

I_{LB}

W24–W21

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W24×62	TFL	0	911	3300	3430	3560	3700	3840	3990	4140	4300	4460	4620	4790
(1550)	2	0.148	807	3200	3320	3440	3570	3710	3840	3990	4130	4280	4440	4600
	3	0.295	703	3080	3190	3300	3420	3550	3680	3810	3940	4080	4230	4380
	4	0.443	600	2940	3040	3150	3260	3370	3490	3610	3730	3860	3990	4130
	BFL	0.590	496	2790	2880	2970	3070	3170	3270	3380	3490	3600	3720	3840
	6	3.46	362	2550	2620	2700	2770	2850	2940	3020	3110	3200	3300	3390
	7	6.57	228	2250	2300	2360	2410	2470	2530	2590	2650	2710	2780	2850
W24×55	TFL	0	810	2890	3000	3120	3240	3370	3500	3630	3770	3910	4060	4210
(1350)	2	0.126	722	2800	2910	3020	3130	3250	3370	3500	3630	3760	3900	4040
	3	0.253	633	2700	2800	2900	3010	3120	3230	3350	3470	3600	3730	3860
	4	0.379	545	2590	2680	2770	2870	2970	3080	3190	3300	3410	3530	3650
	BFL	0.505	456	2460	2540	2630	2710	2800	2900	2990	3090	3200	3300	3410
	6	3.45	329	2240	2300	2370	2440	2520	2590	2670	2750	2830	2910	3000
	7	6.66	203	1970	2010	2060	2110	2160	2210	2260	2320	2380	2440	2500
W21×73	TFL	0	1070	3310	3450	3590	3740	3890	4050	4220	4390	4560	4740	4930
(1600)	2	0.185	921	3180	3310	3440	3580	3720	3870	4020	4180	4340	4510	4680
	3	0.370	767	3030	3140	3260	3380	3510	3650	3780	3930	4070	4220	4380
	4	0.555	614	2840	2940	3050	3160	3270	3390	3510	3630	3760	3890	4030
	BFL	0.740	460	2630	2710	2790	2880	2980	3070	3170	3270	3380	3490	3600
	6	2.61	364	2470	2540	2610	2680	2760	2840	2930	3010	3100	3190	3290
	7	4.72	269	2290	2340	2400	2460	2520	2580	2650	2720	2790	2860	2940
W21×68	TFL	0	1000	3060	3190	3320	3460	3600	3750	3910	4060	4230	4400	4570
(1480)	2	0.171	860	2940	3060	3180	3310	3440	3580	3720	3870	4020	4180	4340
	3	0.343	719	2800	2910	3020	3140	3260	3380	3510	3640	3780	3920	4060
	4	0.514	577	2640	2730	2830	2930	3030	3140	3250	3370	3490	3610	3740
	BFL	0.685	436	2440	2520	2600	2680	2770	2860	2950	3050	3150	3250	3350
	6	2.59	343	2290	2350	2420	2490	2560	2640	2720	2800	2880	2970	3060
	7	4.74	251	2110	2170	2220	2270	2330	2390	2450	2520	2580	2650	2720
W21×62	TFL	0	913	2760	2870	2990	3120	3250	3380	3520	3670	3810	3970	4120
(1330)	2	0.154	786	2650	2760	2870	2980	3100	3230	3360	3490	3630	3770	3920
	3	0.308	659	2520	2620	2720	2830	2940	3050	3170	3290	3410	3540	3670
	4	0.461	533	2380	2470	2560	2650	2740	2840	2950	3050	3160	3270	3390
	BFL	0.615	406	2210	2280	2350	2430	2510	2590	2680	2770	2860	2950	3050
	6	2.57	317	2060	2120	2190	2250	2320	2390	2460	2530	2610	2690	2770
	7	4.79	228	1900	1950	1990	2040	2100	2150	2210	2260	2320	2380	2450

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

I_{LB}

W21

Table 3–20 (continued)

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W21×57	TFL	0	837	2480	2590	2700	2820	2940	3060	3190	3320	3460	3600	3740
(1170)	2	0.163	730	2390	2490	2600	2710	2820	2930	3050	3170	3300	3430	3570
	3	0.325	624	2290	2380	2480	2580	2680	2790	2890	3010	3120	3240	3370
	4	0.488	517	2170	2260	2340	2430	2520	2610	2710	2810	2920	3020	3130
	BFL	0.650	411	2040	2110	2180	2260	2330	2420	2500	2590	2680	2770	2860
	6	2.87	310	1880	1940	2000	2060	2130	2190	2260	2330	2410	2480	2560
	7	5.36	209	1700	1740	1780	1830	1880	1930	1980	2030	2080	2140	2200
W21×55	TFL	0	810	2390	2490	2590	2710	2820	2940	3060	3190	3320	3450	3590
(1140)	2	0.131	703	2300	2390	2490	2590	2700	2810	2930	3040	3160	3290	3420
	3	0.261	596	2200	2280	2370	2470	2560	2660	2770	2880	2990	3100	3220
	4	0.392	488	2080	2150	2230	2320	2400	2490	2580	2680	2780	2880	2980
	BFL	0.522	381	1940	2000	2070	2140	2210	2290	2370	2450	2530	2620	2710
	6	2.62	292	1800	1850	1910	1970	2030	2090	2160	2230	2290	2370	2440
	7	5.00	203	1640	1680	1720	1770	1810	1860	1910	1960	2010	2070	2120
W21×50	TFL	0	736	2120	2210	2310	2410	2510	2620	2730	2850	2970	3090	3220
(984)	2	0.134	648	2050	2130	2220	2320	2420	2520	2620	2730	2840	2960	3070
	3	0.268	561	1970	2050	2130	2220	2310	2400	2500	2600	2700	2810	2910
	4	0.401	474	1870	1950	2020	2100	2180	2270	2350	2440	2540	2630	2730
	BFL	0.535	386	1760	1830	1900	1960	2040	2110	2190	2270	2350	2430	2520
	6	2.91	285	1620	1670	1720	1780	1840	1900	1960	2020	2090	2160	2230
	7	5.58	184	1440	1470	1510	1550	1600	1640	1680	1730	1780	1830	1880
W21×48	TFL	0	707	2030	2120	2210	2310	2410	2510	2620	2730	2840	2960	3080
(959)	2	0.108	619	1960	2040	2130	2220	2310	2410	2510	2610	2710	2820	2940
	3	0.215	532	1880	1960	2030	2120	2200	2290	2380	2480	2570	2670	2780
	4	0.323	444	1790	1850	1930	2000	2080	2160	2240	2320	2410	2500	2590
	BFL	0.430	357	1680	1740	1800	1860	1930	2000	2070	2140	2220	2300	2380
	6	2.69	267	1550	1590	1650	1700	1750	1810	1870	1930	1990	2050	2120
	7	5.26	177	1390	1420	1460	1500	1540	1580	1620	1670	1710	1760	1810
W21×44	TFL	0	649	1830	1910	2000	2080	2180	2270	2370	2470	2570	2680	2790
(843)	2	0.113	576	1770	1850	1930	2010	2100	2190	2280	2370	2470	2570	2680
	3	0.225	503	1700	1780	1850	1930	2010	2090	2170	2260	2350	2450	2550
	4	0.338	430	1630	1690	1760	1830	1910	1980	2060	2140	2220	2310	2400
	BFL	0.450	357	1540	1600	1660	1720	1790	1860	1930	2000	2070	2150	2230
	6	2.92	259	1410	1450	1500	1550	1600	1660	1710	1770	1830	1890	1950
	7	5.69	162	1240	1270	1300	1340	1380	1410	1450	1490	1530	1580	1620

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 3–20 (continued)

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

I_{LB}

W18

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W18×60	TFL	0	882	2070	2170	2280	2390	2500	2620	2740	2870	3000	3140	3280
(984)	2	0.174	750	1990	2080	2170	2270	2380	2490	2600	2720	2840	2960	3090
	3	0.348	619	1880	1970	2050	2140	2240	2330	2430	2540	2650	2760	2870
	4	0.521	488	1760	1830	1910	1980	2070	2150	2240	2330	2420	2520	2620
	BFL	0.695	357	1610	1670	1730	1790	1860	1930	2000	2070	2150	2220	2300
	6	2.17	288	1520	1570	1620	1680	1730	1790	1850	1910	1980	2040	2110
	7	3.81	220	1420	1460	1500	1540	1590	1640	1690	1740	1790	1840	1900
W18×55	TFL	0	810	1880	1970	2070	2170	2270	2380	2490	2610	2730	2850	2980
(890)	2	0.158	691	1800	1890	1970	2070	2160	2260	2360	2470	2580	2690	2810
	3	0.315	573	1710	1790	1870	1950	2030	2120	2220	2310	2410	2510	2620
	4	0.473	454	1600	1670	1740	1810	1880	1960	2040	2120	2210	2300	2390
	BFL	0.630	336	1470	1520	1580	1640	1700	1760	1830	1900	1970	2040	2110
	6	2.16	269	1380	1430	1480	1530	1580	1630	1690	1750	1810	1870	1930
	7	3.86	202	1290	1320	1360	1400	1440	1480	1530	1580	1620	1670	1720
W18×50	TFL	0	733	1690	1770	1850	1940	2040	2140	2240	2340	2450	2560	2680
(800)	2	0.143	626	1620	1690	1770	1850	1940	2030	2120	2220	2320	2420	2530
	3	0.285	520	1540	1600	1680	1750	1830	1910	1990	2080	2170	2260	2360
	4	0.428	413	1440	1500	1560	1620	1690	1760	1830	1910	1990	2070	2150
	BFL	0.570	306	1320	1370	1420	1470	1530	1590	1650	1710	1770	1840	1900
	6	2.10	245	1240	1290	1330	1370	1420	1470	1520	1570	1630	1680	1740
	7	3.83	183	1150	1190	1220	1260	1290	1330	1370	1420	1460	1500	1550
W18×46	TFL	0	677	1540	1610	1690	1770	1860	1950	2040	2140	2240	2340	2450
(712)	2	0.151	585	1480	1550	1620	1700	1780	1860	1950	2040	2130	2230	2320
	3	0.303	494	1410	1470	1540	1610	1680	1760	1840	1920	2000	2090	2180
	4	0.454	402	1330	1380	1440	1500	1570	1640	1700	1780	1850	1930	2010
	BFL	0.605	310	1230	1280	1330	1380	1430	1490	1550	1610	1670	1740	1800
	6	2.37	240	1140	1180	1220	1270	1310	1360	1410	1460	1510	1570	1620
	7	4.33	169	1040	1070	1100	1140	1170	1210	1240	1280	1320	1360	1410
W18×40	TFL	0	588	1320	1380	1450	1520	1600	1680	1760	1840	1930	2020	2110
(612)	2	0.131	509	1270	1330	1390	1460	1530	1600	1670	1750	1830	1910	2000
	3	0.263	430	1210	1260	1320	1380	1450	1510	1580	1650	1720	1800	1880
	4	0.394	351	1140	1190	1240	1290	1350	1410	1470	1530	1590	1660	1730
	BFL	0.525	272	1060	1100	1140	1190	1240	1280	1340	1390	1440	1500	1560
	6	2.29	210	983	1020	1050	1090	1130	1170	1210	1260	1300	1350	1400
	7	4.28	147	894	920	948	977	1010	1040	1070	1100	1140	1170	1210

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[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

I_{LB}
W18–W16

Table 3–20 (continued)
Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W18×35 (510)	TFL	0	515	1120	1170	1230	1300	1360	1430	1500	1570	1650	1720	1800
	2	0.106	451	1080	1130	1190	1240	1300	1370	1430	1500	1570	1640	1720
	3	0.213	387	1030	1080	1130	1180	1240	1300	1360	1420	1480	1550	1620
	4	0.319	323	977	1020	1070	1120	1170	1220	1270	1330	1390	1450	1510
	BFL	0.425	260	917	955	995	1040	1080	1130	1170	1220	1270	1320	1380
	6	2.38	194	842	873	906	940	975	1010	1050	1090	1130	1170	1220
	7	4.56	129	753	776	800	825	851	878	906	935	965	996	1030
W16×45 (586)	TFL	0	663	1260	1330	1400	1470	1550	1630	1720	1810	1900	1990	2090
	2	0.141	564	1200	1270	1330	1400	1470	1550	1630	1710	1790	1880	1970
	3	0.283	464	1140	1200	1250	1320	1380	1450	1520	1590	1670	1740	1830
	4	0.424	365	1060	1110	1160	1220	1270	1330	1390	1450	1520	1580	1650
	BFL	0.565	266	971	1010	1050	1090	1140	1190	1230	1280	1340	1390	1450
	6	1.81	216	916	950	985	1020	1060	1100	1140	1190	1230	1280	1330
	7	3.26	166	855	882	911	941	973	1010	1040	1070	1110	1150	1190
W16×40 (518)	TFL	0	589	1110	1170	1230	1300	1370	1440	1510	1590	1670	1760	1840
	2	0.126	501	1060	1120	1170	1230	1300	1360	1430	1510	1580	1660	1740
	3	0.253	412	1000	1050	1110	1160	1220	1280	1340	1400	1470	1540	1610
	4	0.379	324	936	979	1020	1070	1120	1170	1230	1280	1340	1400	1460
	BFL	0.505	236	855	890	926	964	1000	1040	1090	1130	1180	1230	1280
	6	1.73	191	807	836	867	900	934	969	1010	1040	1080	1120	1170
	7	3.18	147	754	778	803	829	857	886	916	947	979	1010	1050
W16×36 (448)	TFL	0	529	969	1020	1080	1140	1200	1260	1330	1400	1470	1550	1630
	2	0.108	453	929	979	1030	1090	1140	1200	1260	1330	1390	1460	1540
	3	0.215	378	883	927	975	1020	1080	1130	1180	1240	1300	1370	1430
	4	0.323	303	828	867	908	951	996	1040	1090	1140	1200	1250	1310
	BFL	0.430	228	762	795	829	864	901	940	981	1020	1070	1110	1160
	6	1.82	180	713	740	769	799	830	863	897	932	969	1010	1050
	7	3.45	132	656	678	700	724	749	774	801	829	858	888	919
W16×31 (375)	TFL	0	456	826	872	921	972	1030	1080	1140	1200	1260	1330	1390
	2	0.110	396	794	837	882	930	979	1030	1080	1140	1200	1260	1320
	3	0.220	335	757	796	837	881	926	973	1020	1070	1130	1180	1240
	4	0.330	274	713	748	785	823	863	905	948	993	1040	1090	1140
	BFL	0.440	213	662	692	723	755	789	824	861	900	939	981	1020
	6	1.99	164	613	638	664	691	719	748	779	811	844	878	914
	7	3.79	114	555	574	593	614	635	657	680	705	729	755	782

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 3–20 (continued)

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

I_{LB}

W16–W14

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W16×26	TFL	0	384	673	712	753	795	840	886	935	985	1040	1090	1150
(301)	2	0.0863	337	649	685	723	763	805	848	893	940	989	1040	1090
	3	0.173	289	621	654	689	726	764	804	845	888	933	980	1030
	4	0.259	242	589	619	650	683	718	754	791	830	870	912	955
	BFL	0.345	194	551	577	604	633	663	694	726	760	795	832	869
	6	2.04	145	505	526	549	572	596	622	648	676	705	734	765
	7	4.00	96	450	465	482	499	517	535	554	575	595	617	640
W14×38	TFL	0	558	842	894	949	1010	1070	1130	1200	1260	1340	1410	1490
(385)	2	0.129	471	803	851	901	954	1010	1070	1130	1190	1260	1320	1390
	3	0.258	384	758	800	845	891	941	992	1050	1100	1160	1220	1280
	4	0.386	297	703	739	777	817	858	902	948	996	1050	1100	1150
	BFL	0.515	209	634	662	692	723	756	791	827	864	903	943	985
	6	1.42	174	602	627	653	680	709	739	770	803	837	872	909
	7	2.55	140	568	589	611	634	658	684	710	738	766	796	827
W14×34	TFL	0	500	744	790	839	890	944	1000	1060	1120	1180	1250	1320
(340)	2	0.114	423	710	753	797	844	894	945	999	1050	1110	1170	1240
	3	0.228	347	671	709	749	791	835	881	929	979	1030	1090	1140
	4	0.341	270	623	656	690	726	764	803	844	887	932	978	1030
	BFL	0.455	193	565	591	618	646	676	708	740	774	810	847	885
	6	1.41	159	535	557	581	605	631	659	687	716	747	779	812
	7	2.60	125	502	520	540	560	582	604	628	652	677	704	731
W14×30	TFL	0	442	643	683	726	771	818	868	919	973	1030	1090	1150
(291)	2	0.0963	378	615	653	692	734	777	823	870	920	972	1030	1080
	3	0.193	313	583	616	652	689	728	769	812	857	903	951	1000
	4	0.289	248	544	573	604	636	670	706	743	781	822	863	907
	BFL	0.385	183	497	521	546	572	600	629	659	690	723	757	793
	6	1.48	147	467	487	508	531	554	579	605	631	659	688	719
	7	2.82	111	432	448	466	484	503	522	543	565	587	611	635
W14×26	TFL	0	385	554	589	626	666	707	750	795	842	891	942	994
(245)	2	0.105	332	531	564	598	635	673	713	754	798	843	890	939
	3	0.210	279	504	534	565	598	633	669	707	747	788	830	875
	4	0.315	226	473	500	527	556	587	619	652	687	723	760	799
	BFL	0.420	174	437	459	482	507	533	559	587	617	647	679	712
	6	1.67	135	405	424	443	463	485	507	531	555	580	607	634
	7	3.18	96.1	368	382	397	413	430	447	465	484	503	523	544

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

LB
W14–W12

Table 3–20 (continued)
Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W14×22	TFL	0	325	455	484	515	548	583	619	657	696	737	780	824
(199)	2	0.0838	283	437	465	494	524	556	590	625	662	700	740	781
	3	0.168	241	417	442	469	496	526	557	589	622	657	694	731
	4	0.251	199	393	416	439	464	491	518	547	577	608	640	674
	BFL	0.335	157	366	385	405	427	449	473	497	523	550	577	606
	6	1.69	119	336	352	369	386	405	424	444	466	487	510	534
	7	3.34	81.2	301	313	326	339	352	367	382	398	414	431	449
W12×30	TFL	0	440	532	569	608	649	693	739	787	837	889	944	1000
(238)	2	0.110	368	505	539	575	613	652	694	738	784	831	881	933
	3	0.220	296	474	504	536	569	604	641	679	720	762	806	852
	4	0.330	225	437	462	488	516	546	577	609	643	679	716	754
	BFL	0.440	153	390	409	429	450	473	496	521	547	574	602	632
	6	1.12	131	373	390	408	427	447	468	490	513	537	562	588
	7	1.94	110	355	370	386	403	420	438	458	478	499	520	543
W12×26	TFL	0	382	455	487	521	557	595	634	676	719	764	812	861
(204)	2	0.0950	321	434	463	494	526	561	597	635	674	716	759	804
	3	0.190	259	407	433	460	489	520	552	585	620	657	695	735
	4	0.285	197	375	397	420	444	470	497	525	554	585	617	651
	BFL	0.380	136	336	353	371	389	409	430	452	474	498	523	549
	6	1.08	116	321	336	352	368	386	404	424	444	465	487	510
	7	1.95	95.6	305	317	331	345	360	376	393	410	428	447	467
W12×22	TFL	0	324	372	399	428	458	490	524	559	596	635	675	717
(156)	2	0.106	281	356	381	408	437	466	498	531	565	601	638	677
	3	0.213	238	339	362	386	412	439	468	498	529	562	596	631
	4	0.319	196	318	339	361	384	408	433	460	488	517	547	579
	BFL	0.425	153	294	312	330	350	370	392	415	438	463	489	516
	6	1.66	117	270	285	300	316	333	351	370	390	410	431	454
	7	3.04	81	242	253	265	277	290	303	317	332	347	364	380
W12×19	TFL	0	279	312	335	360	386	413	442	472	504	537	571	607
(130)	2	0.0875	244	300	322	345	369	394	421	449	479	510	542	575
	3	0.175	209	286	306	327	349	373	397	423	450	479	508	539
	4	0.263	174	270	288	307	327	348	370	393	417	443	469	497
	BFL	0.350	139	251	267	283	300	318	338	358	379	401	424	447
	6	1.65	104	229	242	255	269	284	300	316	333	351	370	389
	7	3.12	69.7	203	212	222	232	243	255	267	280	293	306	321

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.
[b] $Y2$ = distance from top of the steel beam to concrete flange force.
[c] See Figure 3–3c for PNA locations.
[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

Table 3–20 (continued)

Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

I_{LB}

W12–W10

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W12×16	TFL	0	236	254	273	294	316	339	363	388	415	442	471	501
(103)	2	0.0663	209	245	263	282	303	324	347	371	396	422	449	477
	3	0.133	183	235	252	270	289	309	330	352	375	399	425	451
	4	0.199	156	223	238	255	272	291	310	330	351	373	396	420
	BFL	0.265	130	210	224	239	254	270	288	306	324	344	365	386
	6	1.70	94.4	189	200	212	225	238	251	266	281	297	313	331
	7	3.32	58.9	163	171	179	188	197	207	217	228	239	250	262
W12×14	TFL	0	208	220	237	255	274	295	316	338	361	386	411	438
(88.6)	2	0.0563	185	212	229	246	264	283	303	324	345	368	392	417
	3	0.113	163	204	219	235	252	270	288	308	328	350	372	395
	4	0.169	141	195	209	223	239	255	272	290	309	329	349	371
	BFL	0.225	118	184	196	209	223	238	253	269	286	304	322	341
	6	1.69	85.2	165	175	186	197	208	221	234	247	261	276	291
	7	3.36	51.9	141	148	155	163	171	179	188	198	207	217	228
W10×26	TFL	0	381	339	368	398	430	464	500	537	577	619	662	708
(144)	2	0.110	317	322	347	375	404	435	467	501	537	575	615	656
	3	0.220	254	300	323	347	373	400	429	459	491	524	559	595
	4	0.330	190	274	293	313	334	357	381	406	432	460	489	519
	BFL	0.440	127	242	256	271	287	304	322	341	361	381	403	426
	6	0.898	111	232	245	259	273	288	305	322	339	358	378	398
	7	1.51	95.1	222	233	245	258	272	286	301	317	334	351	369
W10×22	TFL	0	324	281	304	330	357	386	416	448	481	516	553	591
(118)	2	0.0900	273	267	289	312	336	363	390	419	450	482	516	551
	3	0.180	221	250	269	290	312	335	359	385	413	441	471	502
	4	0.270	169	230	246	263	282	301	322	344	367	391	416	443
	BFL	0.360	117	204	217	230	245	260	276	293	311	329	349	369
	6	0.953	99.2	194	205	217	230	243	258	273	288	305	322	340
	7	1.71	81.1	183	193	203	214	225	237	250	263	277	292	307
W10×19	TFL	0	281	239	259	282	305	330	356	384	413	444	476	509
(96.3)	2	0.0988	241	228	247	267	289	312	336	362	389	417	447	477
	3	0.198	201	215	232	251	270	291	313	336	361	386	413	440
	4	0.296	162	200	216	232	249	267	286	307	328	350	374	398
	BFL	0.395	122	183	195	208	223	238	254	270	288	307	326	346
	6	1.28	96.1	169	179	191	203	215	229	243	258	273	290	307
	7	2.31	70.2	153	162	170	180	190	200	211	223	235	248	261

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

I_{LB}
W10

Table 3–20 (continued)
Lower Bound Elastic Moment of Inertia, I_{LB}, for Plastic Composite Sections

Shape[d]	PNA[c]	$Y1$[a]	ΣQ_n	$Y2$[b], in.										
		in.	kip	2	2.5	3	3.5	4	4.5	5	5.5	6	6.5	7
W10×17	TFL	0	250	206	224	244	265	287	310	334	360	387	415	445
(81.9)	2	0.0825	217	197	214	233	252	272	294	317	341	366	392	419
	3	0.165	183	187	202	219	236	255	275	295	317	340	364	389
	4	0.248	150	175	189	203	219	236	253	271	291	311	332	354
	BFL	0.330	117	161	173	185	199	213	227	243	259	277	295	313
	6	1.31	89.8	148	157	168	179	190	202	215	229	243	258	274
	7	2.46	62.4	132	139	147	155	164	173	183	193	204	215	227
W10×15	TFL	0	221	177	193	210	228	248	268	290	312	336	361	387
(68.9)	2	0.0675	194	170	185	201	218	236	255	275	296	319	342	366
	3	0.135	167	162	176	191	206	223	240	259	278	299	320	343
	4	0.203	140	153	165	179	193	208	223	240	258	276	295	316
	BFL	0.270	113	142	153	165	177	190	204	218	234	250	267	284
	6	1.35	83.8	128	137	147	157	167	179	190	203	216	230	244
	7	2.60	55.1	112	118	125	133	140	148	157	166	176	186	196
W10×12	TFL	0	177	139	152	165	180	195	211	228	246	265	285	306
(53.8)	2	0.0525	156	134	145	158	172	186	201	217	234	252	271	290
	3	0.105	135	127	138	150	163	176	190	205	221	237	254	272
	4	0.158	114	120	130	141	152	164	177	191	205	220	235	251
	BFL	0.210	93.6	113	121	131	141	152	163	175	187	200	214	228
	6	1.31	68.9	101	109	116	124	133	142	152	162	172	183	195
	7	2.61	44.2	87.8	92.9	98.3	104	110	117	124	131	138	146	155

[a] $Y1$ = distance from top of the steel beam to plastic neutral axis.

[b] $Y2$ = distance from top of the steel beam to concrete flange force.

[c] See Figure 3–3c for PNA locations.

[d] Value in parentheses is I_x (in.^{4}) of non-composite steel shape.

F_u = 65 ksi

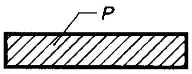
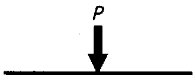
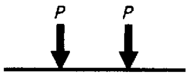
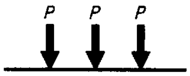
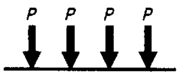
Table 3–21
Shear Stud
Nominal Horizontal Shear for One Stud, Q_n, kip

Q_N

Deck condition			Stud diameter, in.	Normal weight concrete		Light weight concrete	
				w_c = 145 pcf		w_c = 110 pcf	
				f'_c = 3 ksi	f'_c = 4 ksi	f'_c = 3 ksi	f'_c = 4 ksi
No deck			3/8	5.26	6.53	4.28	5.31
			1/2	9.35	11.6	7.60	9.43
			5/8	14.6	18.1	11.9	14.7
			3/4	21.0	26.1	17.1	21.2
Deck Parallel	$\frac{w_r}{h_r} \geq 1.5$		3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
	$\frac{w_r}{h_r} < 1.5$		3/8	4.58	4.58	4.28	4.58
			1/2	8.14	8.14	7.60	8.14
			5/8	12.7	12.7	11.9	12.7
			3/4	18.3	18.3	17.1	18.3
Deck Perpendicular	Weak studs per rib	1	3/8	4.31	4.31	4.28	4.31
			1/2	7.66	7.66	7.66	7.66
			5/8	12.0	12.0	12.0	12.0
			3/4	17.2	17.2	17.2	17.2
		2	3/8	3.66	3.66	3.66	3.66
			1/2	6.51	6.51	6.51	6.51
			5/8	10.2	10.2	10.2	10.2
			3/4	14.6	14.6	14.6	14.6
		3	3/8	3.02	3.02	3.02	3.02
			1/2	5.36	5.36	5.36	5.36
			5/8	8.38	8.38	8.38	8.38
			3/4	12.1	12.1	12.1	12.1
	Strong studs per rib	1	3/8	5.26	5.38	4.28	5.31
			1/2	9.35	9.57	7.60	9.43
			5/8	14.6	15.0	11.9	14.7
			3/4	21.0	21.5	17.1	21.2
		2	3/8	4.58	4.58	4.28	4.58
			1/2	8.14	8.14	7.60	8.14
			5/8	12.7	12.7	11.9	12.7
			3/4	18.3	18.3	17.1	18.3
		3	3/8	3.77	3.77	3.77	3.77
			1/2	6.70	6.70	6.70	6.70
			5/8	10.5	10.5	10.5	10.5
			3/4	15.1	15.1	15.1	15.1

Note:
Tabulated values are applicable only to concrete made with ASTM C33 aggregates.
After-weld shear stud lengths assumed to be ≥ Deck height + 1.5 in.

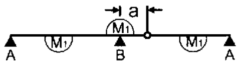
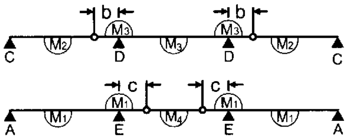
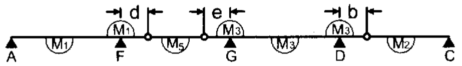
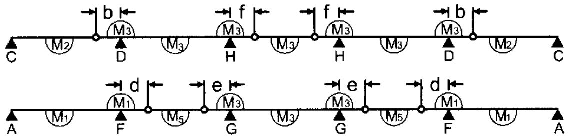
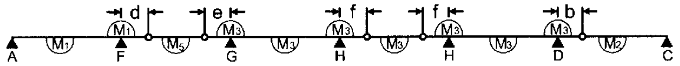
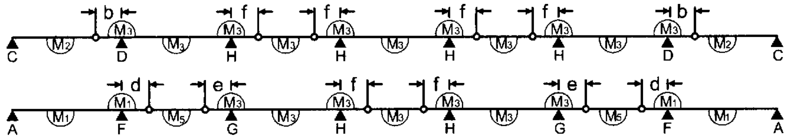
Table 3–22a
Concentrated Load Equivalents

n	Loading	Coeff.	Simple Beam	Beam Fixed One End, Supported at Other	Beam Fixed Both Ends
∞		a	0.125	0.070	0.042
		b	—	0.125	0.083
		c	0.500	0.375	—
		d	—	0.625	0.500
		e	0.013	0.005	0.003
		f	1.000	1.000	0.667
		g	1.000	0.415	0.300
2		a	0.250	0.156	0.125
		b	—	0.188	0.125
		c	0.500	0.313	—
		d	—	0.688	0.500
		e	0.021	0.009	0.005
		f	2.000	1.500	1.000
		g	0.800	0.477	0.400
3		a	0.333	0.222	0.111
		b	—	0.333	0.222
		c	1.000	0.667	—
		d	—	1.333	1.000
		e	0.036	0.015	0.008
		f	2.667	2.667	1.778
		g	1.022	0.438	0.333
4		a	0.500	0.266	0.188
		b	—	0.469	0.313
		c	1.500	1.031	—
		d	—	1.969	1.500
		e	0.050	0.021	0.010
		f	4.000	3.750	2.500
		g	0.950	0.428	0.320
5		a	0.600	0.360	0.200
		b	—	0.600	0.400
		c	2.000	1.400	—
		d	—	2.600	2.000
		e	0.063	0.027	0.013
		f	4.800	4.800	3.200
		g	1.008	0.424	0.312

Maximum positive moment (kip-ft): aPL
Maximum negative moment (kip-ft): bPL
Pinned end reaction (kips): cP
Fixed end reaction (kips): dP
Maximum deflection (in.): ePl^3/EI

Equivalent simple span uniform load (kips): fP
Deflection coefficient for equivalent simple span uniform load: g
Number of equal load spaces: n
Span of beam (ft): L
Span of beam (in.): l

Table 3–22b
Beam Diagrams and Formulas
Design Properties of Cantilevered Beams – Equal Loads, Equally Spaced

No. Spans	System
2	
3	
4	
5	
≥6 (even)	
≥7 (odd)	

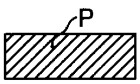
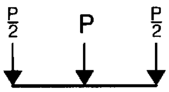
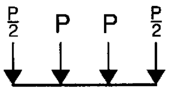
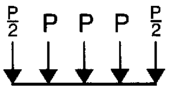
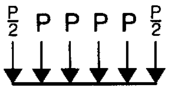
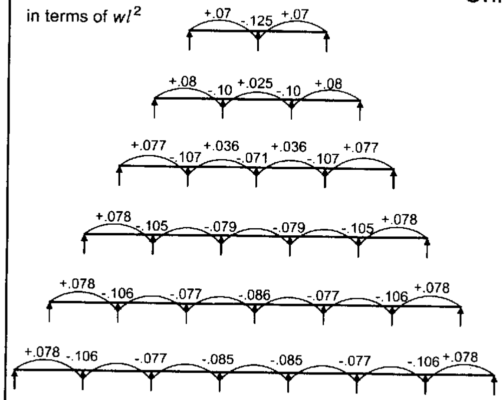
	n	∞	2	3	4	5
Typical Span Loading						
Moments	M1	0.086×PL	0.167×PL	0.250×PL	0.333×PL	0.429×PL
	M2	0.096×PL	0.188×PL	0.278×PL	0.375×PL	0.480×PL
	M3	0.063×PL	0.125×PL	0.167×PL	0.250×PL	0.300×PL
	M4	0.039×PL	0.083×PL	0.083×PL	0.167×PL	0.171×PL
	M5	0.051×PL	0.104×PL	0.139×PL	0.208×PL	0.249×PL
Reactions	A	0.414×P	0.833×P	1.250×P	1.667×P	2.071×P
	B	1.172×P	2.333×P	3.500×P	4.667×P	5.857×P
	C	0.438×P	0.875×P	1.333×P	1.750×P	2.200×P
	D	1.063×P	2.125×P	3.167×P	4.250×P	5.300×P
	E	1.086×P	2.167×P	3.250×P	4.333×P	5.429×P
	F	1.109×P	2.208×P	3.333×P	4.417×P	5.557×P
	G	0.977×P	1.958×P	2.917×P	3.917×P	4.871×P
	H	1.000×P	2.000×P	3.000×P	4.000×P	5.000×P
Cantilever Dimensions	a	0.172×L	0.250×L	0.200×L	0.182×L	0.176×L
	b	0.125×L	0.200×L	0.143×L	0.143×L	0.130×L
	c	0.220×L	0.333×L	0.250×L	0.222×L	0.229×L
	d	0.204×L	0.308×L	0.231×L	0.211×L	0.203×L
	e	0.157×L	0.273×L	0.182×L	0.176×L	0.160×L
	f	0.147×L	0.250×L	0.167×L	0.167×L	0.150×L

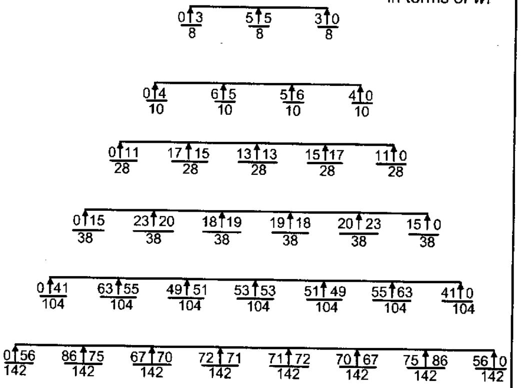
Table 3–22c
Continuous Beams
Moments and Shear Coefficients – Equal Spans, Equally Loaded

Uniform Load

Moment in terms of wl^2

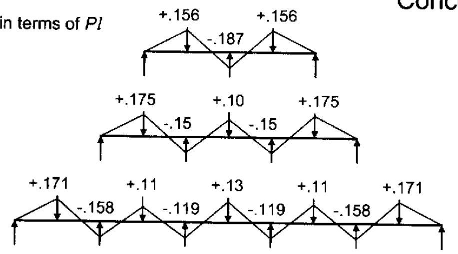


Shear in terms of wl

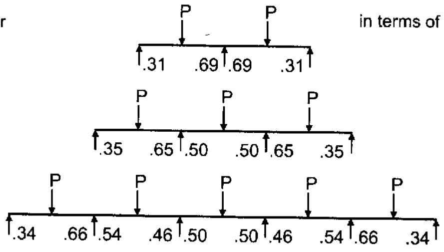


Concentrated Loads
at center

Moment in terms of Pl

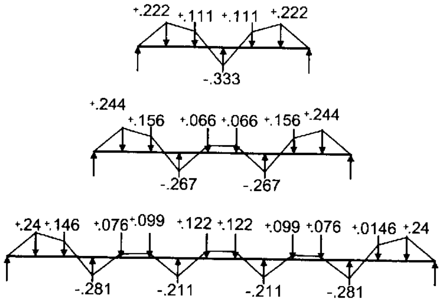


Shear in terms of P

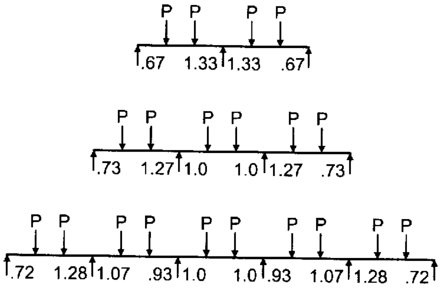


Concentrated Loads
at third points

Moment in terms of Pl

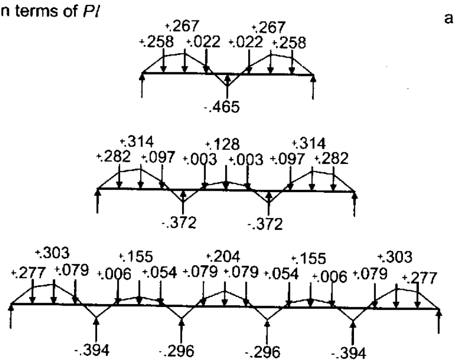


Shear in terms of P



Concentrated Loads
at quarter points

Moment in terms of Pl



Shear in terms of P

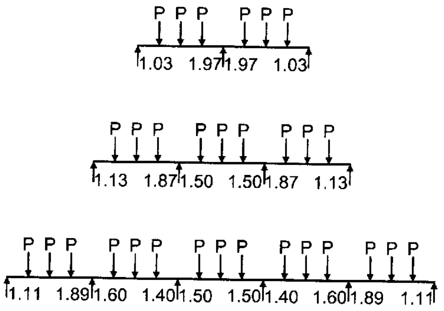
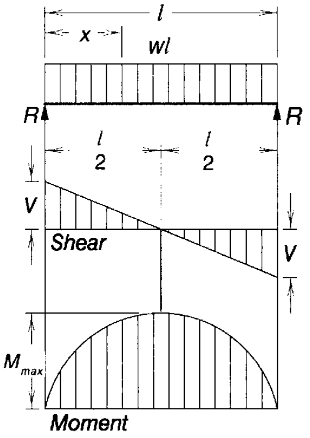


Table 3–23
Shears, Moments, and Deflections

1. SIMPLE BEAM — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= wl$

$R = V$ .... $= \frac{wl}{2}$

V_x .... $= w\left(\frac{l}{2} - x\right)$

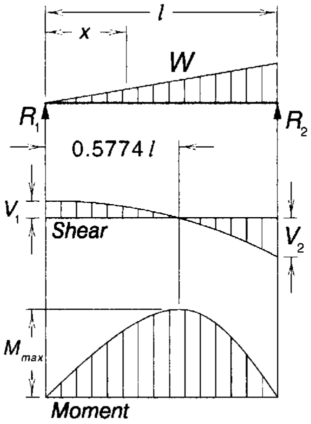
M_{max} (at center) .... $= \frac{wl^2}{8}$

M_x .... $= \frac{wx}{2}(l - x)$

Δ_{max} (at center) .... $= \frac{5wl^4}{384EI}$

Δ_x .... $= \frac{wx}{24EI}\left(l^3 - 2lx^2 + x^3\right)$

2. SIMPLE BEAM — LOAD INCREASING UNIFORMLY TO ONE END



Total Equiv. Uniform Load .... $= \frac{16W}{9\sqrt{3}} = 1.03\,W$

$R_1 = V_1$ .... $= \frac{W}{3}$

$R_2 = V_2 = V_{max}$ .... $= \frac{2W}{3}$

V_x .... $= \frac{W}{3} - \frac{Wx^2}{l^2}$

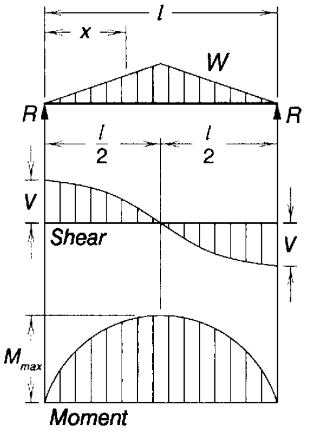
M_{max} $\left(\text{at } x = \frac{l}{\sqrt{3}} = 0.557\,l\right)$ .... $= \frac{2Wl}{9\sqrt{3}} = 0.128\,Wl$

M_x .... $= \frac{Wx}{3l^2}(l^2 - x^2)$

Δ_{max} $\left(\text{at } x = l\sqrt{1 - \sqrt{\frac{8}{15}}} = 0.519\,l\right)$ .... $= 0.0130\frac{Wl^3}{EI}$

Δ_x .... $= \frac{Wx}{180EIl^2}\left(3x^4 - 10l^2x^2 + 7l^4\right)$

3. SIMPLE BEAM — LOAD INCREASING UNIFORMLY TO CENTER



Total Equiv. Uniform Load .... $= \frac{4W}{3}$

$R = V$ .... $= \frac{W}{2}$

V_x (when $x < \frac{l}{2}$) .... $= \frac{W}{2l^2}\left(l^2 - 4x^2\right)$

M_{max} (at center) .... $= \frac{Wl}{6}$

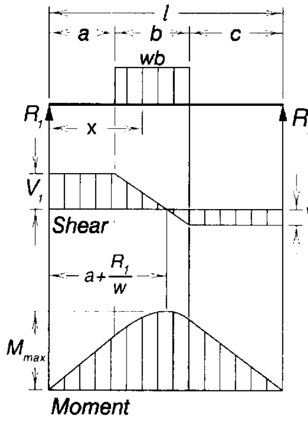
M_x (when $x < \frac{l}{2}$) .... $= Wx\left(\frac{1}{2} - \frac{2x^2}{3l^2}\right)$

Δ_{max} (at center) .... $= \frac{Wl^3}{60EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{Wx}{480EIl^2}\left(5l^2 - 4x^2\right)^2$

Table 3–23 (continued)
Shears, Moments, and Deflections

4. SIMPLE BEAM — UNIFORM LOAD PARTIALLY DISTRIBUTED



$R_1 = V_1$ (max. when $a < c$) …………… $= \frac{wb}{2l}(2c+b)$

$R_2 = V_2$ (max. when $a > c$) …………… $= \frac{wb}{2l}(2a+b)$

V_x (when $x > a$ and $< (a+b)$) …. $= R_1 - w(x-a)$

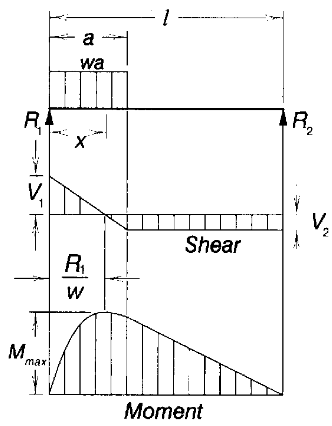
M_{max} $\left(\text{at } x = a + \frac{R_1}{w}\right)$ …………………… $= R_1\left(a + \frac{R_1}{2w}\right)$

M_x (when $x < a$) …………………… $= R_1 x$

M_x (when $x > a$ and $< (a+b)$) …. $= R_1 x - \frac{w}{2}(x-a)^2$

M_x (when $x > (a+b)$) ……………… $= R_2(l-x)$

5. SIMPLE BEAM — UNIFORM LOAD PARTIALLY DISTRIBUTED AT ONE END



$R_1 = V_1 = V_{max}$ ………………………………… $= \frac{wa}{2l}(2l-a)$

$R_2 = V_2$ ……………………………………………… $= \frac{wa^2}{2l}$

V_x (when $x < a$) ………………………… $= R_1 - wx$

M_{max} $\left(\text{at } x = \frac{R_1}{w}\right)$ ………………………… $= \frac{R_1^2}{2w}$

M_x (when $x < a$) ………………………… $= R_1 x - \frac{wx^2}{2}$

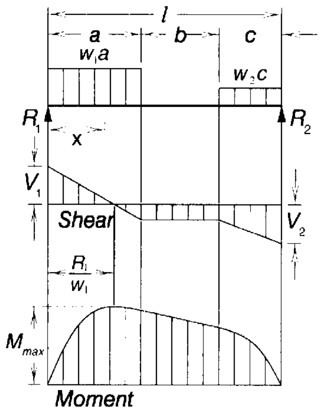
M_x (when $x > a$) ………………………… $= R_2(l-x)$

Δ_x (when $x < a$) ………………………… $= \frac{wx}{24EIl}\left(a^2(2l-a)^2 - 2ax^2(2l-a) + lx^3\right)$

Δ_x (when $x > a$) ………………………… $= \frac{wa^2(l-x)}{24EIl}\left(4xl - 2x^2 - a^2\right)$

6. SIMPLE BEAM — UNIFORM LOAD PARTIALLY DISTRIBUTED AT EACH END

$R_1 = V_1$ ……………………………………… $= \frac{w_1 a(2l-a) + w_2 c^2}{2l}$



$R_2 = V_2$ ……………………………………… $= \frac{w_2 c(2l-c) + w_1 a^2}{2l}$

V_x (when $x < a$) …………………… $= R_1 - w_1 x$

V_x (when $a < x < (a+b)$) ………… $= R_1 - w_1 a$

V_x (when $x > (a+b)$) ……………… $= R_2 - w_2(l-x)$

M_{max} $\left(\text{at } x = \frac{R_1}{w_1}, \text{when } R_1 < w_1 a\right)$ …. $= \frac{R_1^2}{2w_1}$

M_{max} $\left(\text{at } x = l - \frac{R_2}{w_2}, \text{when } R_2 < w_2 c\right)$ $= \frac{R_2^2}{2w_2}$

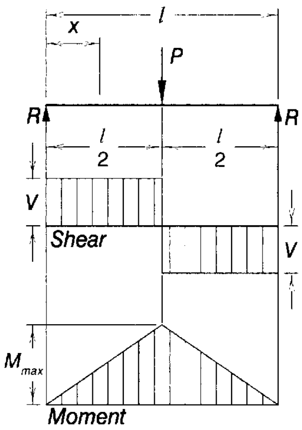
M_x (when $x < a$) …………………… $= R_1 x - \frac{w_1 x^2}{2}$

M_x (when $a < x < (a+b)$) ………… $= R_1 x - \frac{w_1 a}{2}(2x-a)$

M_x (when $x > (a+b)$) ……………… $= R_2(l-x) - \frac{w_2(l-x)^2}{2}$

Table 3–23 (continued)
Shears, Moments, and Deflections

7. SIMPLE BEAM — CONCENTRATED LOAD AT CENTER



Total Equiv. Uniform Load .... $= 2P$

$R = V$ .... $= \frac{P}{2}$

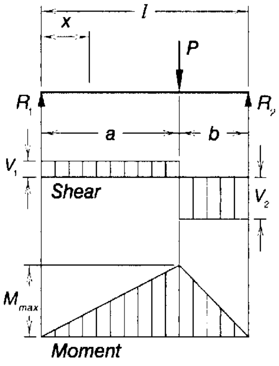
M_{max} (at point of load) .... $= \frac{Pl}{4}$

M_x (when $x < \frac{l}{2}$) .... $= \frac{Px}{2}$

Δ_{max} (at point of load) .... $= \frac{Pl^3}{48EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{Px}{48EI}\left(3l^2 - 4x^2\right)$

8. SIMPLE BEAM — CONCENTRATED LOAD AT ANY POINT



Total Equiv. Uniform Load .... $= \frac{8Pab}{l^2}$

$R_1 = V_1$ ($= V_{max}$ when $a < b$) .... $= \frac{Pb}{l}$

$R_2 = V_2$ ($= V_{max}$ when $a > b$) .... $= \frac{Pa}{l}$

M_{max} (at point of load) .... $= \frac{Pab}{l}$

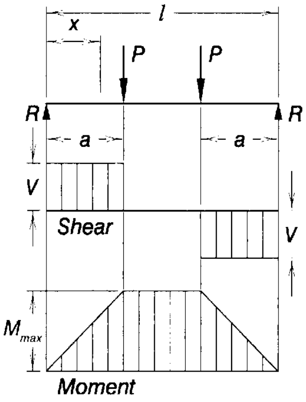
M_x (when $x < a$) .... $= \frac{Pbx}{l}$

Δ_{max} $\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}}, \text{when } a > b\right)$ .... $= \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EIl}$

Δ_a (at point of load) .... $= \frac{Pa^2b^2}{3EIl}$

Δ_x (when $x < a$) .... $= \frac{Pbx}{6EIl}\left(l^2 - b^2 - x^2\right)$

9. SIMPLE BEAM — TWO EQUAL CONCENTRATED LOADS SYMMETRICALLY PLACED



Total Equiv. Uniform Load .... $= \frac{8Pa}{l}$

$R = V$ .... $= P$

M_{max} (between loads) .... $= Pa$

M_x (when $x < a$) .... $= Px$

Δ_{max} (at center) .... $= \frac{Pa}{24EI}\left(3l^2 - 4a^2\right)$

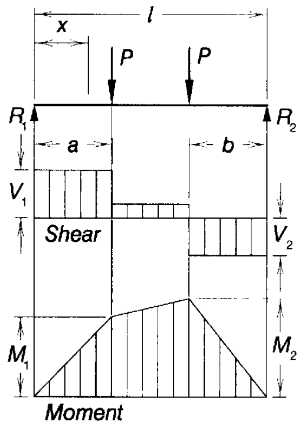
Δ_{max} (when $a = \frac{l}{3}$) .... $= \frac{Pl^3}{28EI}$

Δ_x (when $x < a$) .... $= \frac{Px}{6EI}\left(3la - 3a^2 - x^2\right)$

Δ_x (when $a < x < (l - a)$) .... $= \frac{Pa}{6EI}\left(3lx - 3x^2 - a^2\right)$

Table 3–23 (continued)
Shears, Moments, and Deflections

10. SIMPLE BEAM — TWO EQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ ($= V_{max}$ when $a < b$) .... $= \frac{P}{l}(l - a + b)$

$R_2 = V_2$ ($= V_{max}$ when $a > b$) .... $= \frac{P}{l}(l - b + a)$

V_x (when $a < x < (l - b)$) .... $= \frac{P}{l}(b - a)$

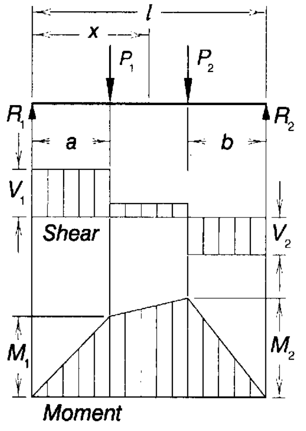
M_1 ($= M_{max}$ when $a > b$) .... $= R_1 a$

M_2 ($= M_{max}$ when $a < b$) .... $= R_2 b$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $a < x < (l - b)$) .... $= R_1 x - P(x - a)$

11. SIMPLE BEAM — TWO UNEQUAL CONCENTRATED LOADS UNSYMMETRICALLY PLACED



$R_1 = V_1$ .... $= \dfrac{P_1(l - a) + P_2 b}{l}$

$R_2 = V_2$ .... $= \dfrac{P_1 a + P_2(l - b)}{l}$

V_x (when $a < x < (l - b)$) .... $= R_1 - P_1$

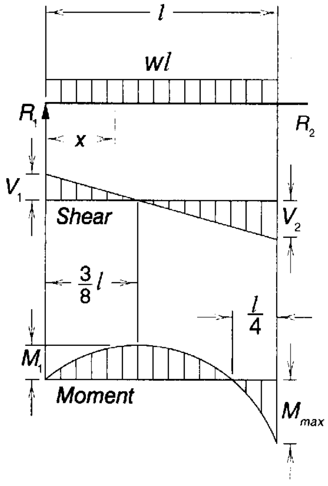
M_1 ($= M_{max}$ when $R_1 < P_1$) .... $= R_1 a$

M_2 ($= M_{max}$ when $R_2 < P_2$) .... $= R_2 b$

M_x (when $x < a$) .... $= R_1 x$

M_x (when $a < x < (l - b)$) .... $= R_1 x - P_1(x - a)$

12. BEAM FIXED AT ONE END, SUPPORTED AT OTHER — UNIFORMLY DISTRUBTED LOAD



Total Equiv. Uniform Load .... $= wl$

$R_1 = V_1$ .... $= \frac{3wl}{8}$

$R_2 = V_2 = V_{max}$ .... $= \frac{5wl}{8}$

V_x .... $= R_1 - wx$

M_{max} .... $= \frac{wl^2}{8}$

M_1 (at $x = \frac{3}{8}l$) .... $= \frac{9}{128}wl^2$

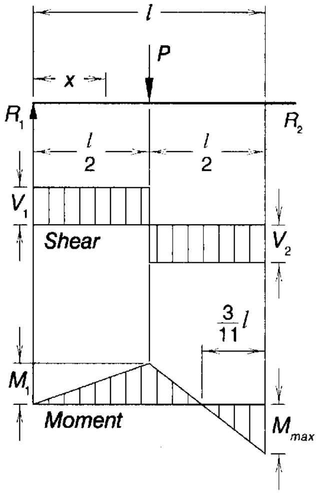
M_x .... $= R_1 x - \frac{wx^2}{2}$

Δ_{max} (at $x = \frac{l}{16}\left(1 + \sqrt{33}\right) = 0.422\,l$) .... $= \frac{wl^4}{185\,EI}$

Δ_x .... $= \frac{wx}{48\,EI}\left(l^3 - 3lx^2 + 2x^3\right)$

Table 3–23 (continued)
Shears, Moments, and Deflections

13. BEAM FIXED AT ONE END, SUPPORTED AT OTHER — CONCENTRATED LOAD AT CENTER



Total Equiv. Uniform Load .... $= \frac{3P}{2}$

$R_1 = V_1$ .... $= \frac{5P}{16}$

$R_2 = V_2 = V_{max}$ .... $= \frac{11P}{16}$

M_{max} (at fixed end) .... $= \frac{3Pl}{16}$

M_1 (at point of load) .... $= \frac{5Pl}{32}$

M_x (at $x < \frac{l}{2}$).. $= \frac{5Px}{16}$

M_x (when $x > \frac{l}{2}$)................................. $= P\left(\frac{l}{2} - \frac{11x}{16}\right)$

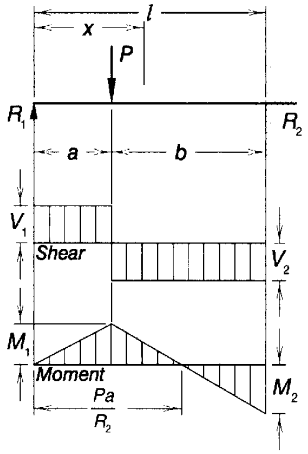
Δ_{max} (at $x = \frac{l}{\sqrt{5}} = 0.447\,l$) .... $= \frac{Pl^3}{48\,EI\sqrt{5}} = 0.00932\frac{Pl^3}{EI}$

Δ_x (at point of load) .... $= \frac{7Pl^3}{768\,EI}$

Δ_x (at $x < \frac{l}{2}$).. $= \frac{Px}{96\,EI}\left(3l^2 - 5x^2\right)$

Δ_x (at $x > \frac{l}{2}$).. $= \frac{P}{96\,EI}(x-l)^2\,(11x-2l)$

14. BEAM FIXED AT ONE END, SUPPORTED AT THE OTHER — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ .... $= \frac{Pb^2}{2l^3}(a+2l)$

$R_2 = V_2$ .... $= \frac{Pa}{2l^3}\left(3l^2 - a^2\right)$

M_1 (at point of load) .... $= R_1 a$

M_2 (at fixed end) .... $= \frac{Pab}{2l^2}(a+l)$

M_x (at $x < a$).. $= R_1 x$

M_x (when $x > a$).................................. $= R_1 x - P(x-a)$

Δ_{max} $\left(\text{when } a < 0.414l \text{ at } x = l\frac{\left(l^2 + a^2\right)}{\left(3l^2 - a^2\right)}\right)$.. $= \frac{Pa}{3\,EI}\frac{\left(l^2 - a^2\right)^3}{\left(3l^2 - a^2\right)^2}$

Δ_{max} $\left(\text{when } a > 0.414l \text{ at } x = l\sqrt{\frac{a}{2l+a}}\right)$ ..... $= \frac{Pab^2}{6\,EI}\sqrt{\frac{a}{2l+a}}$

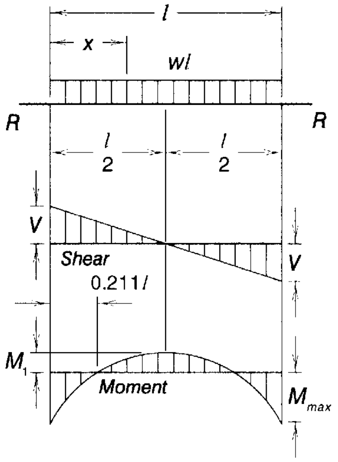
Δ_a (at point of load) .... $= \frac{Pa^2b^3}{12\,EIl^3}(3l+a)$

Δ_x (when $x < a$).................................. $= \frac{Pb^2x}{12\,EIl^3}\left(3al^2 - 2lx^2 - ax^2\right)$

Δ_x (when $x > a$).................................. $= \frac{Pa}{12\,EIl^3}(l-x)^2\left(3l^2x - a^2x - 2a^2l\right)$

Table 3–23 (continued)
Shears, Moments, and Deflections

15. BEAM FIXED AT BOTH ENDS — UNIFORMLY DISTRIBUTED LOADS



Total Equiv. Uniform Load .... $= \frac{2wl}{3}$

$R = V$ .... $= \frac{wl}{2}$

V_x .... $= w\left(\frac{l}{2} - x\right)$

M_{max} (at ends) .... $= \frac{wl^2}{12}$

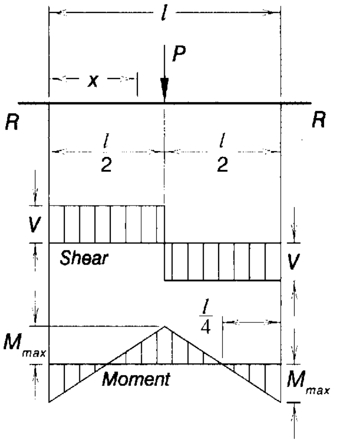
M_1 (at center) .... $= \frac{wl^2}{24}$

M_x .... $= \frac{w}{12}\left(6lx - l^2 - 6x^2\right)$

Δ_{max} (at center) .... $= \frac{wl^4}{384EI}$

Δ_x .... $= \frac{wx^2}{24EI}(l - x)^2$

16. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT CENTER



Total Equiv. Uniform Load .... $= P$

$R = V$ .... $= \frac{P}{2}$

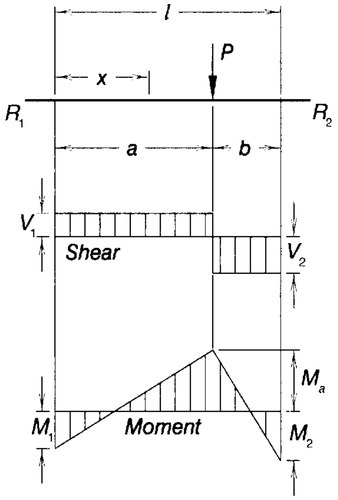
M_{max} (at center and ends) .... $= \frac{Pl}{8}$

M_x (when $x < \frac{l}{2}$) .... $= \frac{P}{8}(4x - l)$

Δ_{max} (at center) .... $= \frac{Pl^3}{192EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{Px^2}{48EI}(3l - 4x)$

17. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1 (= V_{max}$ when $a < b)$ .... $= \frac{Pb^2}{l^3}(3a + b)$

$R_2 = V_2 (= V_{max}$ when $a > b)$ .... $= \frac{Pa^2}{l^3}(a + 3b)$

M_1 ($= M_{max}$ when $a < b$) .... $= \frac{Pab^2}{l^2}$

M_2 ($= M_{max}$ when $a > b$) .... $= \frac{Pa^2b}{l^2}$

M_a (at point of load) .... $= \frac{2Pa^2b^2}{l^3}$

M_x (when $x < a$) .... $= R_1x - \frac{Pab^2}{l^2}$

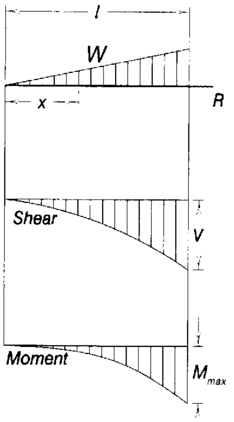
Δ_{max} (when $a > b$ at $x = \frac{2al}{3a + b}$) .... $= \frac{2Pa^3b^2}{3EI(3a + b)^2}$

Δ_a (at point of load) .... $= \frac{Pa^3b^3}{3EIl^3}$

Δ_x (when $x < a$) .... $= \frac{Pb^2x^2}{6EIl^3}(3al - 3ax - bx)$

Table 3–23 (continued)
Shears, Moments, and Deflections

18. CANTILEVERED BEAM — LOAD INCREASING UNIFORMLY TO FIXED END



Total Equiv. Uniform Load .... $= \frac{8}{3}W$

$R = V$ .... $= W$

V_x .... $= W\frac{x^2}{l^2}$

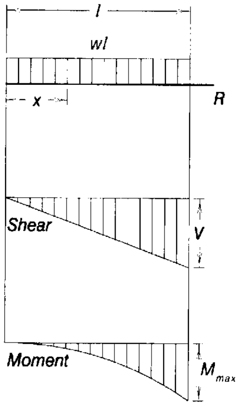
M_{max} (at fixed end) .... $= \frac{Wl}{3}$

M_x .... $= \frac{Wx^3}{3l^2}$

Δ_{max} (at free end) .... $= \frac{Wl^3}{15EI}$

Δ_x .... $= \frac{W}{60EIl^2}\left(x^5 - 5l^4x + 4l^5\right)$

19. CANTILEVERED BEAM — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= 4wl$

$R = V$ .... $= wl$

V_x .... $= wx$

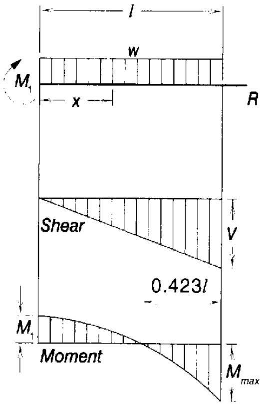
M_{max} (at fixed end) .... $= \frac{wl^2}{2}$

M_x .... $= \frac{wx^2}{2}$

Δ_{max} (at free end) .... $= \frac{wl^4}{8EI}$

Δ_x .... $= \frac{w}{24EI}\left(x^4 - 4l^3x + 3l^4\right)$

20. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER — UNIFORMLY DISTRIBUTED LOAD



Total Equiv. Uniform Load .... $= \frac{8}{3}wl$

$R = V$ .... $= wl$

V_x .... $= wx$

M_1 (at deflected end) .... $= \frac{wl^2}{6}$

M_{max} (at fixed end) .... $= \frac{wl^2}{3}$

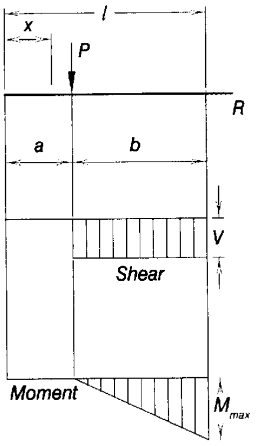
M_x .... $= \frac{w}{6}\left(l^2 - 3x^2\right)$

Δ_{max} (at deflected end) .... $= \frac{wl^4}{24EI}$

Δ_x .... $= \frac{w\left(l^2 - x^2\right)^2}{24EI}$

Table 3–23 (continued)
Shears, Moments, and Deflections

21. CANTILEVERED BEAM — CONCENTRATED LOAD AT ANY POINT



Total Equiv. Uniform Load .... $= \frac{8Pb}{l}$

$R = V$ .... $= P$

M_{max} (at fixed end) .... $= Pb$

M_x (when $x > a$) .... $= P(x - a)$

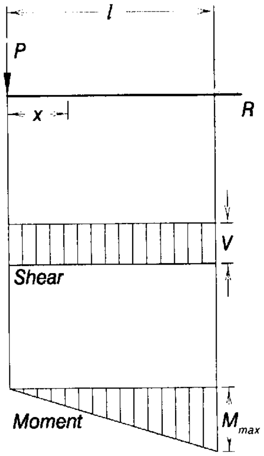
Δ_{max} (at free end) .... $= \frac{Pb^2}{6EI}(3l - b)$

Δ_a (at point of load) .... $= \frac{Pb^3}{3EI}$

Δ_x (when $x < a$) .... $= \frac{Pb^2}{6EI}(3l - 3x - b)$

Δ_x (when $x > a$) .... $= \frac{P(l - x)^2}{6EI}(3b - l + x)$

22. CANTILEVERED BEAM — CONCENTRATED LOAD AT FREE END



Total Equiv. Uniform Load .... $= 8P$

$R = V$ .... $= P$

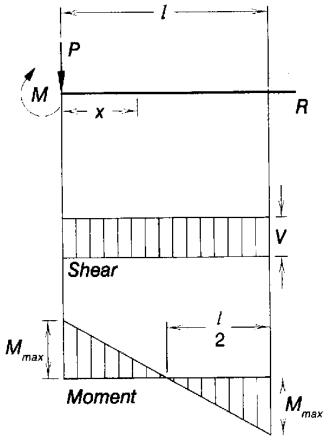
M_{max} (at fixed end) .... $= Pl$

M_x .... $= Px$

Δ_{max} (at free end) .... $= \frac{Pl^3}{3EI}$

Δ_x .... $= \frac{P}{6EI}\left(2l^3 - 3l^2x + x^3\right)$

23. BEAM FIXED AT ONE END, FREE TO DEFLECT VERTICALLY BUT NOT ROTATE AT OTHER — CONCENTRATED LOAD AT DEFLECTED END



Total Equiv. Uniform Load .... $= 4P$

$R=V$ .... $= P$

M_{max} (at both ends) .... $= \frac{Pl}{2}$

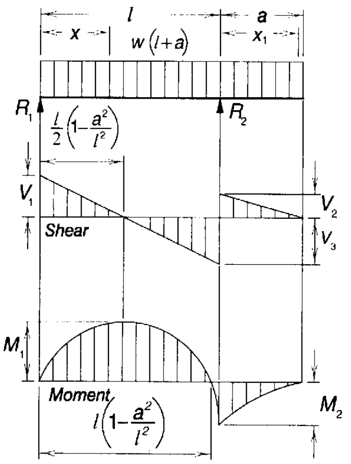
M_x .... $= P\left(\frac{l}{2} - x\right)$

Δ_{max} (at deflected end) .... $= \frac{Pl^3}{12EI}$

Δ_x .... $= \frac{P(l - x)^2}{12EI}(l + 2x)$

Table 3–23 (continued)
Shears, Moments, and Deflections

24. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD



$R_1 = V_1$ .... $= \frac{w}{2l}\left(l^2 - a^2\right)$

$R_2 = V_2 + V_3$ .... $= \frac{w}{2l}(l+a)^2$

V_2 .... $= wa$

V_3 .... $= \frac{w}{2l}\left(l^2 + a^2\right)$

V_x (between supports) .... $= R_1 - wx$

V_{x_1} (for overhang) .... $= w(a - x_1)$

M_1 $\left(\text{at } x = \frac{l}{2}\left[1 - \frac{a^2}{l^2}\right]\right)$ .... $= \frac{w}{8l^2}(l+a)^2(l-a)^2$

M_2 (at R_2) .... $= \frac{wa^2}{2}$

M_x (between supports) .... $= \frac{wx}{2l}\left(l^2 - a^2 - xl\right)$

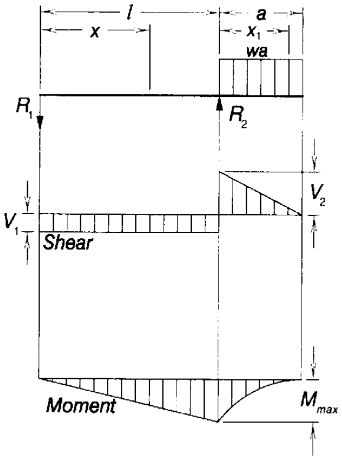
M_{x_1} (for overhang) .... $= \frac{w}{2}(a - x_1)^2$

Δ_x (between supports) .... $= \frac{wx}{24EIl}\left(l^4 - 2l^2x^2 + lx^3 - 2a^2l^2 + 2a^2x^2\right)$

Δ_{x_1} (for overhang) .... $= \frac{wx_1}{24EI}\left(4a^2l - l^3 + 6a^2x_1 - 4ax_1^2 + x_1^3\right)$

NOTE: For a negative value of Δ_x, deflection is upward.

25. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD ON OVERHANG



$R_1 = V_1$ .... $= \frac{wa^2}{2l}$

$R_2 = V_1 + V_2$ .... $= \frac{wa}{2l}(2l + a)$

V_2 .... $= wa$

V_{x_1} (for overhang) .... $= w(a - x_1)$

M_{max} (at R_2) .... $= \frac{wa^2}{2}$

M_x (between supports) .... $= \frac{wa^2x}{2l}$

M_{x_1} (for overhang) .... $= \frac{w}{2}(a - x_1)^2$

Δ_{max} $\left(\text{between supports at } x = \frac{l}{\sqrt{3}}\right) = \frac{wa^2l^2}{18\sqrt{3}EI} = 0.0321\frac{wa^2l^2}{EI}$

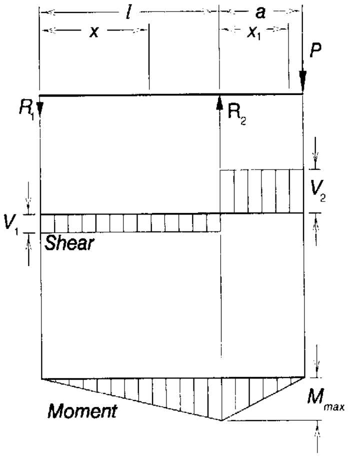
Δ_{max} (for overhang at $x_1 = a$)..... $= \frac{wa^3}{24EI}(4l + 3a)$

Δ_x (between supports) .... $= \frac{wa^2x}{12EIl}\left(l^2 - x^2\right)$

Δ_{x_1} (for overhang) .... $= \frac{wx_1}{24EI}\left(4a^2l + 6a^2x_1 - 4ax_1^2 + x_1^3\right)$

Table 3–23 (continued)
Shears, Moments, and Deflections

26. BEAM OVERHANGING ONE SUPPORT — CONCENTRATED LOAD AT END OF OVERHANG



$R_1 = V_1$ .... $= \frac{Pa}{l}$

$R_2 = V_1 + V_2$ .... $= \frac{P}{l}(l+a)$

V_2 .... $= P$

M_{max} (at R_2) .... $= Pa$

M_x (between supports) .... $= \frac{Pax}{l}$

M_{x_1} (for overhang) .... $= P(a - x_1)$

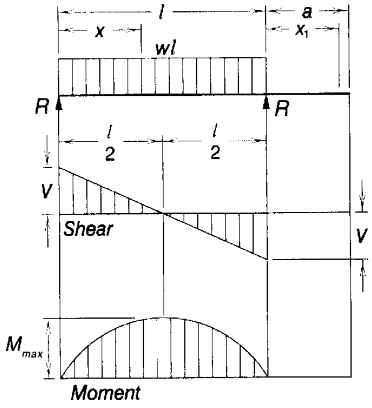
Δ_{max} $\left(\text{between supports at } x = \frac{l}{\sqrt{3}}\right)$ .... $= \frac{Pal^2}{9\sqrt{3}EI} = 0.0642\,\frac{Pal^2}{EI}$

Δ_{max} (for overhang at $x_1 = a$) .... $= \frac{Pa^2}{3EI}(l+a)$

Δ_x (between supports) .... $= \frac{Pax}{6EIl}\left(l^2 - x^2\right)$

Δ_{x_1} (for overhang) .... $= \frac{Px_1}{6EI}\left(2al + 3ax_1 - x_1^2\right)$

27. BEAM OVERHANGING ONE SUPPORT — UNIFORMLY DISTRIBUTED LOAD BETWEEN SUPPORTS



Total Equiv. Uniform Load .... $= wl$

$R = V$ .... $= \frac{wl}{2}$

V_x .... $= w\left(\frac{l}{2} - x\right)$

M_{max} (at center) .... $= \frac{wl^2}{8}$

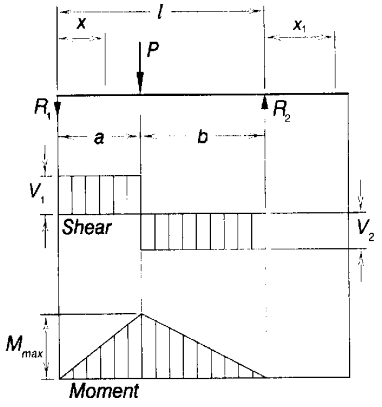
M_x .... $= \frac{wx}{2}(l - x)$

Δ_{max} (at center) .... $= \frac{5wl^4}{384EI}$

Δ_x .... $= \frac{wx}{24EI}\left(l^3 - 2lx^2 + x^3\right)$

Δ_{x_1} .... $= \frac{wl^3x_1}{24EI}$

28. BEAM OVERHANGING ONE SUPPORT — CONCENTRATED LOAD AT ANY POINT BETWEEN SUPPORTS



Total Equiv. Uniform Load .... $= \frac{8Pab}{l^2}$

$R_1 = V_1$ ($= V_{max}$ when $a < b$) .... $= \frac{Pb}{l}$

$R_2 = V_2$ ($= V_{max}$ when $a > b$) .... $= \frac{Pa}{l}$

M_{max} (at point of load) .... $= \frac{Pab}{l}$

M_x (when $x < a$) .... $= \frac{Pbx}{l}$

Δ_{max} $\left(\text{at } x = \sqrt{\frac{a(a+2b)}{3}} \text{ when } a > b\right)$ .... $= \frac{Pab(a+2b)\sqrt{3a(a+2b)}}{27EIl}$

Δ_a (at point of load) .... $= \frac{Pa^2b^2}{3EIl}$

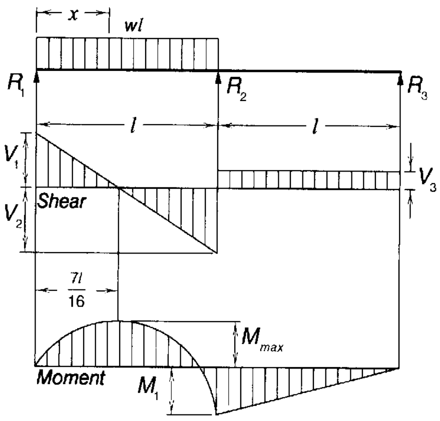
Δ_x (when $x < a$) .... $= \frac{Pbx}{6EIl}\left(l^2 - b^2 - x^2\right)$

Δ_x (when $x > a$) .... $= \frac{Pa(l-x)}{6EIl}\left(2lx - x^2 - a^2\right)$

Δ_{x_1} .... $= \frac{Pabx_1}{6EIl}(l+a)$

Table 3–23 (continued)
Shears, Moments, and Deflections

29. CONTINUOUS BEAM — TWO EQUAL SPANS — UNIFORM LOAD ON ONE SPAN



Total Equiv. Uniform Load .... $= \frac{49}{64} wl$

$R_1 = V_1$ .... $= \frac{7}{16} wl$

$R_2 = V_2 + V_3$ .... $= \frac{5}{8} wl$

$R_3 = V_3$ .... $= -\frac{1}{16} wl$

V_2 .... $= \frac{9}{16} wl$

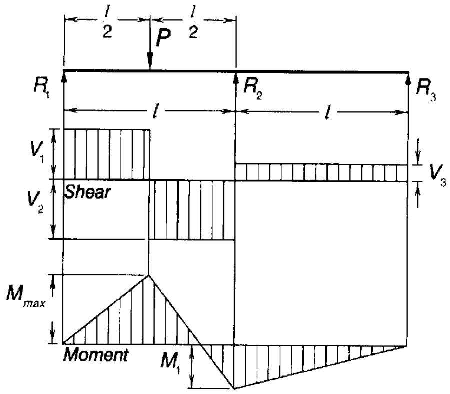
M_{max} (at $x = \frac{7}{16} l$) .... $= \frac{49}{512} wl^2$

M_1 (at support R_2) .... $= \frac{1}{16} wl^2$

M_x (when $x < l$) .... $= \frac{wx}{16}(7l - 8x)$

Δ_{max} (at 0.472 l from R_1) .... $= \frac{0.0092\, wl^4}{EI}$

30. CONTINUOUS BEAM — TWO EQUAL SPANS — CONCENTRATED LOAD AT CENTER OF ONE SPAN



Total Equiv. Uniform Load .... $= \frac{13}{8} P$

$R_1 = V_1$ .... $= \frac{13}{32} P$

$R_2 = V_2 + V_3$ .... $= \frac{11}{16} P$

$R_3 = V_3$ .... $= -\frac{3}{32} P$

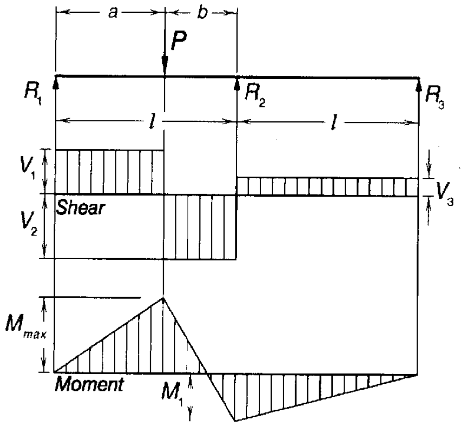
V_2 .... $= \frac{19}{32} P$

M_{max} (at point of load) .... $= \frac{13}{64} Pl$

M_1 (at support R_2) .... $= \frac{3}{32} Pl$

Δ_{max} (at 0.480 l from R_1) .... $= \frac{0.015\, Pl^3}{EI}$

31. CONTINUOUS BEAM — TWO EQUAL SPANS — CONCENTRATED LOAD AT ANY POINT



$R_1 = V_1$ .... $= \frac{Pb}{4l^3}\left(4l^2 - a(l + a)\right)$

$R_2 = V_2 + V_3$ .... $= \frac{Pa}{2l^3}\left(2l^2 + b(l + a)\right)$

$R_3 = V_3$ .... $= -\frac{Pab}{4l^3}(l + a)$

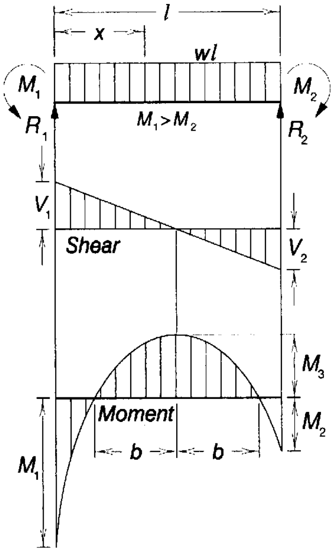
V_2 .... $= \frac{Pa}{4l^3}\left(4l^2 + b(l + a)\right)$

M_{max} (at point of load) .... $= \frac{Pab}{4l^3}\left(4l^2 - a(l + a)\right)$

M_1 (at support R_2) .... $= \frac{Pab}{4l^2}(l + a)$

Table 3–23 (continued)
Shears, Moments, and Deflections

32. BEAM — UNIFORMLY DISTRIBUTED LOAD AND VARIABLE END MOMENTS



$R_1 = V_1$ …………………… $= \frac{wl}{2} + \frac{M_1 - M_2}{l}$

$R_2 = V_2$ …………………… $= \frac{wl}{2} - \frac{M_1 - M_2}{l}$

V_x …………………… $= w\left(\frac{l}{2} - x\right) + \frac{M_1 - M_2}{l}$

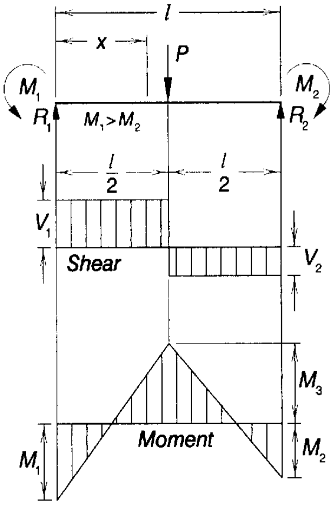
M_3 (at $x = \frac{l}{2} + \frac{M_1 - M_2}{wl}$) …………………… $= \frac{wl^2}{8} - \frac{M_1 + M_2}{2} + \frac{(M_1 - M_2)^2}{2wl^2}$

M_x …………………… $= \frac{wx}{2}(l - x) + \left(\frac{M_1 - M_2}{l}\right)x - M_1$

b (to locate inflection points) …………………… $= \sqrt{\frac{l^2}{4} - \left(\frac{M_1 + M_2}{w}\right) + \left(\frac{M_1 - M_2}{wl}\right)^2}$

$$\Delta_x = \frac{wx}{24EI}\left[x^3 - \left(2l + \frac{4M_1}{wl} - \frac{4M_2}{wl}\right)x^2 + \frac{12M_1}{w}x + l^3 - \frac{8M_1 l}{w} - \frac{4M_2 l}{w}\right]$$

33. BEAM — CONCENTRATED LOAD AT CENTER AND VARIABLE END MOMENTS



$R_1 = V_1$ …………………… $= \frac{P}{2} + \frac{M_1 - M_2}{l}$

$R_2 = V_2$ …………………… $= \frac{P}{2} - \frac{M_1 - M_2}{l}$

M_3 (at center) …………………… $= \frac{Pl}{4} - \frac{M_1 + M_2}{2}$

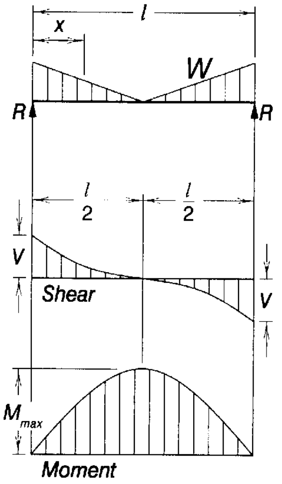
M_x (when $x < \frac{l}{2}$) …………………… $= \left(\frac{P}{2} + \frac{M_1 - M_2}{l}\right)x - M_1$

M_x (when $x > \frac{l}{2}$) …………………… $= \frac{P}{2}(l - x) + \frac{(M_1 - M_2)x}{l} - M_1$

Δ_x (when $x < \frac{l}{2}$) $= \frac{Px}{48EI}\left(3l^2 - 4x^2 - \frac{8(l - x)}{Pl}\left[M_1(2l - x) + M_2(l + x)\right]\right)$

Table 3–23 (continued)
Shears, Moments, and Deflections

34. SIMPLE BEAM — LOAD INCREASING UNIFORMLY FROM CENTER



Total Equiv. Uniform Load .... $= \frac{2W}{3}$

$R = V$ .... $= \frac{W}{2}$

V_x (when $x < \frac{l}{2}$) .... $= \frac{W}{2}\left(\frac{l-2x}{l}\right)^2$

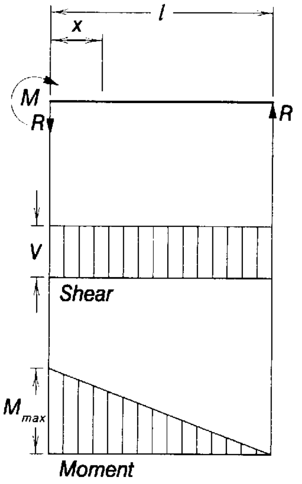
M_{max} (at center) .... $= \frac{Wl}{12}$

M_x (when $x < \frac{l}{2}$) .... $= \frac{W}{2}\left(x - \frac{2x^2}{l} + \frac{4x^3}{3l^2}\right)$

Δ_{max} (at center) .... $= \frac{3Wl^3}{320EI}$

Δ_x (when $x < \frac{l}{2}$) .... $= \frac{W}{12EI}\left(x^3 - \frac{x^4}{l} + \frac{2x^5}{5l^2} - \frac{3l^2x}{8}\right)$

35. SIMPLE BEAM — CONCENTRATED MOMENT AT END



Total Equiv. Uniform Load .... $= \frac{8M}{l}$

$R = V$ .... $= \frac{M}{l}$

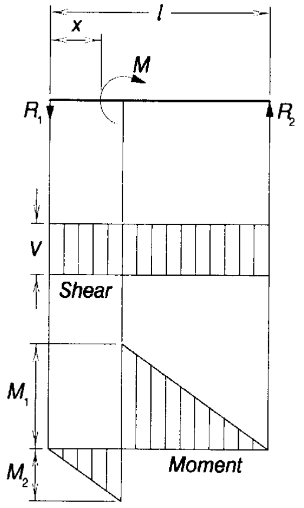
M_{max} .... $= M$

M_x .... $= M\left(1 - \frac{x}{l}\right)$

Δ_{max} (at $x = 0.423\,l$) .... $= 0.0642\frac{Ml^2}{EI}$

Δ_x .... $= \frac{M}{6EI}\left(3x^2 - \frac{x^3}{l} - 2lx\right)$

36.SIMPLE BEAM — CONCENTRATED MOMENT AT ANY POINT



Total Equiv. Uniform Load .... $= \frac{8M}{l}$

$R = V$ .... $= \frac{M}{l}$

M_x (when $x < a$) .... $= Rx$

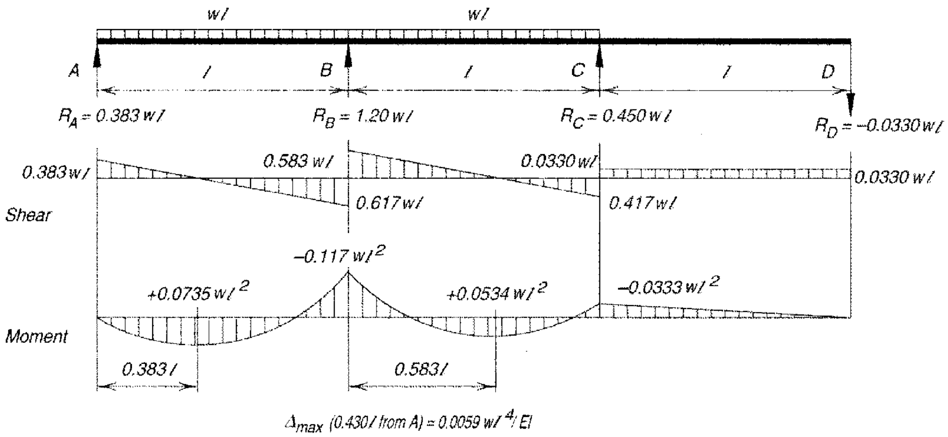
M_x (when $x > a$) .... $= M + Rx$

Δ_x (when $x < a$) .... $= \frac{M}{6EI}\left[\left(6a - \frac{3a^2}{l} - 2l\right)x - \frac{x^3}{l}\right]$

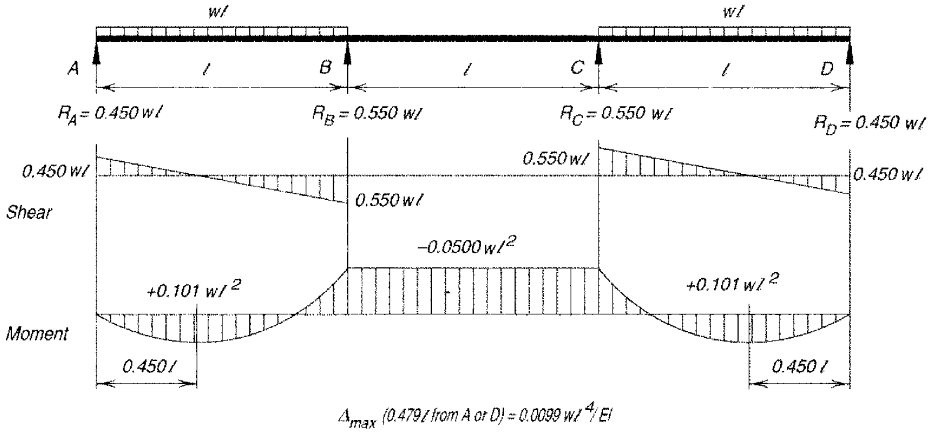
Δ_x .... $= \frac{M}{6EI}\left[3\left(a^2 + x^2\right) - \frac{x^3}{l} - \left(2l + \frac{3a^2}{l}\right)x\right]$

Table 3–23 (continued)
Shears, Moments, and Deflections

37. CONTINUOUS BEAM—THREE EQUAL SPANS—ONE END SPAN UNLOADED



38. CONTINUOUS BEAM—THREE EQUAL SPANS—END SPANS LOADED



39. CONTINUOUS BEAM—THREE EQUAL SPANS—ALL SPANS LOADED

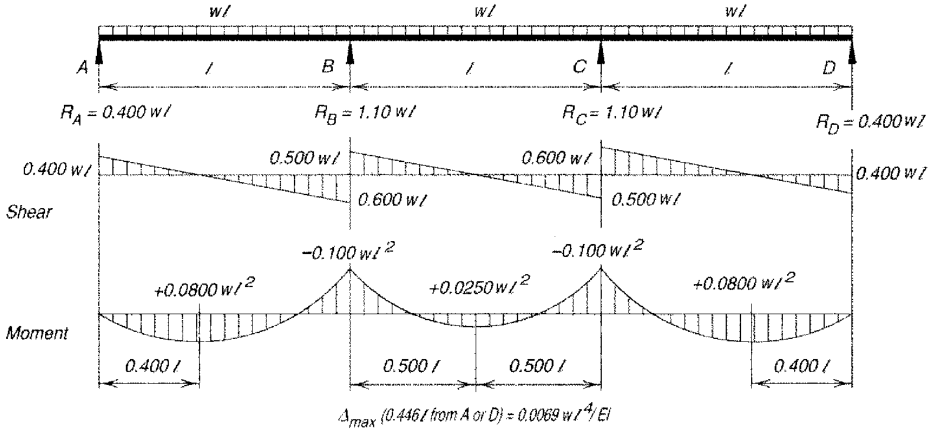
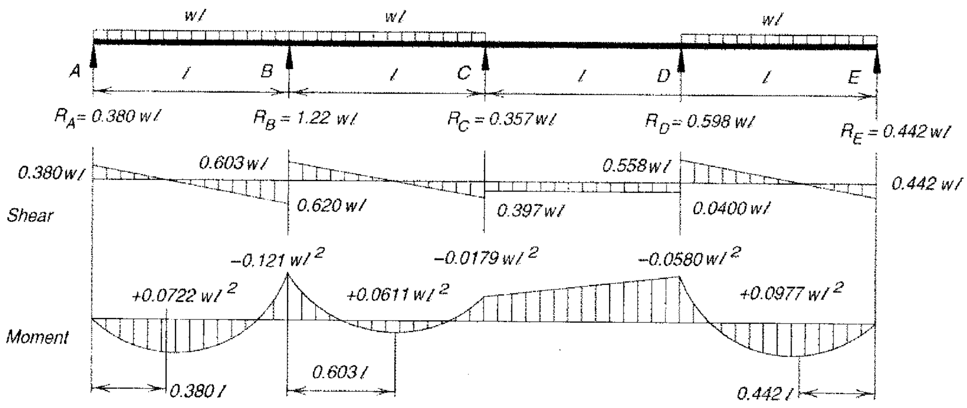


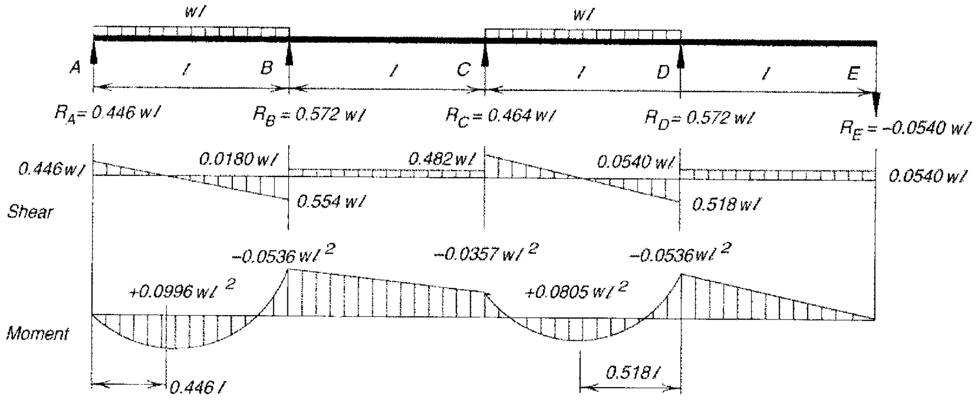
Table 3–23 (continued)
Shears, Moments, and Deflections

40. CONTINUOUS BEAM—FOUR EQUAL SPANS—THIRD SPAN UNLOADED



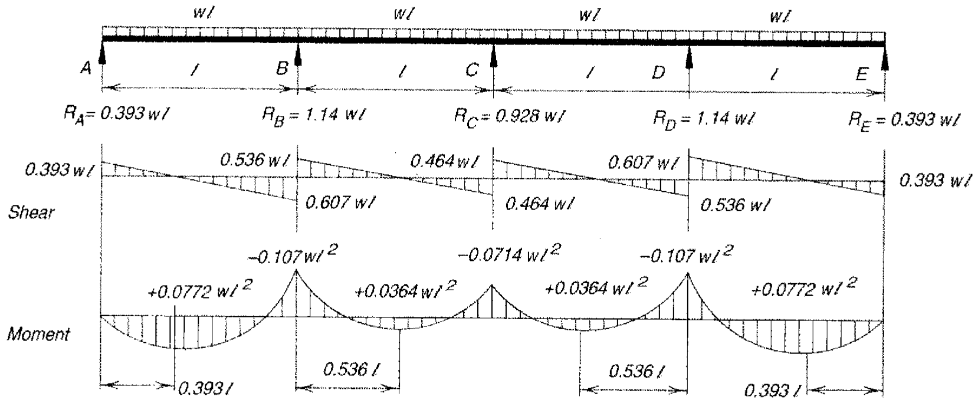
Δ_{max} (0.475 l from E) = 0.0094 wl^4/EI

41. CONTINUOUS BEAM—FOUR EQUAL SPANS—LOAD FIRST AND THIRD SPANS



Δ_{max} (0.477 l from A) = 0.0097 wl^4/EI

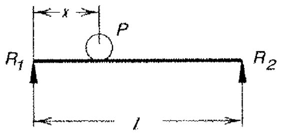
42. CONTINUOUS BEAM—FOUR EQUAL SPANS—ALL SPANS LOADED



Δ_{max} (0.440 l from A and E) = 0.0065 wl^4/EI

Table 3–23 (continued)
Shears, Moments, and Deflections

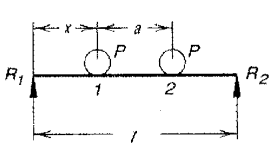
43. SIMPLE BEAM—ONE CONCENTRATED MOVING LOAD



$$R_{1\,\max} = V_{1\,\max}(\text{at } x = 0) \ldots\ldots = P$$

$$M_{\max}\left(\text{at point of load, when } x = \frac{l}{2}\right) \ldots\ldots = \frac{Pl}{4}$$

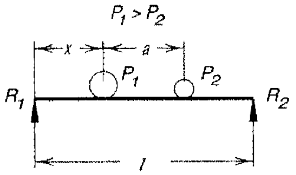
44. SIMPLE BEAM—TWO EQUAL CONCENTRATED MOVING LOADS



$$R_{1\,\max} = V_{1\,\max}(\text{at } x = 0) \ldots\ldots = P\left(2 - \frac{a}{l}\right)$$

$$M_{\max}\begin{cases}\left[\begin{array}{l}\text{when } a < (2-\sqrt{2})l = 0.586l \\ \text{under load 1 at } x = \frac{1}{2}\left(l - \frac{a}{2}\right)\end{array}\right] \ldots\ldots = \frac{P}{2l}\left(l - \frac{a}{2}\right)^2 \\ \left[\begin{array}{l}\text{when } a > (2-\sqrt{2})l = 0.586l \\ \text{with one load at center of span (Case 43)}\end{array}\right] \ldots\ldots = \frac{Pl}{4}\end{cases}$$

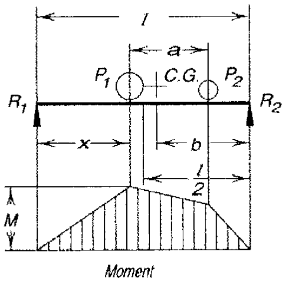
45. SIMPLE BEAM—TWO UNEQUAL CONCENTRATED MOVING LOADS



$$R_{1\,\max} = V_{1\,\max}(\text{at } x = 0) \ldots\ldots = P_1 + P_2\frac{l-a}{l}$$

$$M_{\max}\begin{cases}\left[\text{under } P_1\text{, at } x = \frac{1}{2}\left(l - \frac{P_2 a}{P_1 + P_2}\right)\right] \ldots\ldots = (P_1 + P_2)\frac{x^2}{l} \\ \left[\begin{array}{l}M_{\max} \text{ may occur with larger} \\ \text{load at center of span and other} \\ \text{load off span (Case 43)}\end{array}\right] \ldots\ldots = \frac{P_1 l}{4}\end{cases}$$

GENERAL RULES FOR SIMPLE BEAMS CARRYING MOVING CONCENTRATED LOADS



The maximum shear due to moving concentrated loads occurs at one support when one of the loads is at that support. With several moving loads, the location that will produce maximum shear must be determined by trial.

The maximum bending moment produced by moving concentrated loads occurs under one of the loads when that load is as far from one support as the center of gravity of all the moving loads on the beam is from the other support.

In the accompanying diagram, the maximum bending moment occurs under load P_1 when $x = b$. It should also be noted that this condition occurs when the centerline of the span is midway between the center of gravity of loads and the nearest concentrated load.

PART 4

DESIGN OF COMPRESSION MEMBERS

SCOPE 4–3

AVAILABLE COMPRESSIVE STRENGTH 4–3

LOCAL BUCKLING 4–3

Determining the Width-Thickness Ratios of the Cross-Section 4–3

Determining the Slenderness of the Cross-Section 4–3

EFFECTIVE LENGTH AND COLUMN SLENDERNESS 4–3

COMPOSITE COMPRESSION MEMBERS 4–4

STEEL COMPRESSION—MEMBER SELECTION TABLES 4–4

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Table 4–2. HP-Shapes in Axial Compression. 4–5

Table 4–3. Rectangular HSS in Axial Compression 4–5

Table 4–4. Square HSS in Axial Compression 4–5

Table 4–5. Round HSS in Axial Compression 4–5

Table 4–6. Pipe in Axial Compression 4–5

Table 4–7. WT-Shapes in Axial Compression 4–5

Table 4–8. Equal-Leg Double Angles in Axial Compression 4–6

Table 4–9. LLBB Double Angles in Axial Compression 4–6

Table 4–10. SLBB Double Angles in Axial Compression 4–6

Table 4–11. Concentrically Loaded Single Angles in Axial Compression 4–6

Table 4–12. Eccentrically Loaded Single Angles in Axial Compression 4–7

COMPOSITE COMPRESSION—MEMBER SELECTION TABLES 4–7

Table 4–13. Rectangular HSS Filled with 4-ksi Normal-Weight Concrete in Axial Compression 4–7

Table 4–14. Square HSS Filled With 4-ksi Normal-Weight Concrete in Axial Compression 4–7

Table 4–15. Rectangular HSS Filled With 5-ksi Normal-Weight Concrete in Axial Compression 4–7

Table 4–16. Square HSS Filled With 5-ksi Normal-Weight Concrete in Axial Compression 4–8

Table 4–17. Round HSS Filled With 4-ksi Normal-Weight Concrete in Axial Compression 4–8

Table 4–18. Round HSS Filled With 5-ksi Normal-Weight Concrete in Axial Compression 4–8
Table 4–19. Pipe Filled With 4-ksi Normal-Weight Concrete in Axial Compression 4–8
Table 4–20. Pipe Filled With 5-ksi Normal-Weight Concrete in Axial Compression 4–8
Table 4–21. Stiffness Reduction Factor τ_a 4–8
Table 4–22. Available Critical Stress for Compression Members 4–8
PART 4 REFERENCES 4–9

SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of members subject to axial compression. For the design of members subject to eccentric compression or combined axial compression and flexure, see Part 6. For compression members that are part of a seismic force resisting system in which the seismic response modification factor, *R*, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

AVAILABLE COMPRESSIVE STRENGTH

The available strength of compression members, ϕP_n or P_n/Ω, which must equal or exceed the required strength, P_u or P_a, respectively, is determined according to AISC Specification Chapter E.

LOCAL BUCKLING

Determining the Width-Thickness Ratios of the Cross-Section

Steel compression members are classified on the basis of the width-thickness ratios of the various elements of the cross-section. The width-thickness ratio is calculated for each element of the cross-section per AISC Specification Section B4.

Determining the Slenderness of the Cross-Section

When the width-thickness ratios of all compression elements are less than λ_r, the cross-section is non-slender, and Q, the reduction factor for slender compression elements (elastic local buckling effects), equals 1.0. When the width-thickness ratio of a compression element is greater than λ_r, the cross-section is a slender-element cross-section and $Q < 1.0$ must be included in the calculation of the available compressive strength. Q is determined per AISC Specification Section E7, and λ_r is determined per AISC Specification Section B4 and Table B4.1.

EFFECTIVE LENGTH AND COLUMN SLENDERNESS

Columns are designed for their slenderness, KL/r, per AISC Specification Section E2. The effective length, KL, is equal to L, the physical length between braced points (see AISC Specification Appendix 6) multiplied by K, which is determined per AISC Specification Section C2. In many cases, the stability provisions in AISC Specification Chapter C and Appendix 7 allow the use of $K = 1$. Otherwise, guidance on the proper selection of a value for K is given in AISC Commentary Section C, including the following:

1. For columns with idealized end conditions, recommended vales of K can be determined from AISC Commentary Table C-C2.1.
2. For columns in braced frames (or steel frames that lean on shear walls or another similar structural system) and compression members in trusses, K is normally taken as unity per AISC Specification Section C1.3a, unless a smaller value can be justified by

analysis. Although the alignment chart in AISC Commentary Figure C-C2.3 (sidesway inhibited—braced frames) could be used for that purpose, it should be noted that the stability bracing provisions in AISC Appendix 6 are based upon the use of $K = 1$.

3. For columns in moment frames, the alignment charts in AISC Commentary Figure C-C2.4 (sidesway uninhibited—moment frames) can be used. Per AISC Commentary Section C2, the stiffness reduction factor, τ_a, can be used in the determination of K for columns controlled by inelastic buckling.

As indicated in the User Note in AISC Specification Section E2, compression-member slenderness, KL/r, should preferably be limited to a maximum of 200. Note that this recommendation does not apply to members that are primarily tension members, but subject to incidental compression under other load combinations.

Further information is available in the SSRC *Guide to Stability Design Criteria for Metal Structures* (Galambos, 1998).

COMPOSITE COMPRESSION MEMBERS

For the design of reinforced-concrete-encased and concrete-filled steel compression members, see AISC Specification Section I2. See also AISC Design Guide No. 6 *Load and Resistance Factor Design of W-Shapes Encased in Concrete* (Griffis, 1992). For further information on composite design and construction, see also Viest et al. (1997).

STEEL COMPRESSION—MEMBER SELECTION TABLES

Table 4–1. W-Shapes in Axial Compression

Available strengths in axial compression are given for W-shapes with $F_y = 50$ ksi (ASTM A992). The tabulated values are given for the effective length with respect to the Y-Y axis $(KL)_y$. However, the effective length with respect to the X-X axis $(KL)_x$ must also be investigated. To determine the available strength in axial compression, the table should be entered at the larger of $(KL)_y$ and $(KL)_{y\,eq}$, where

$$(KL)_{y\,eq} = \frac{(KL)_x}{\dfrac{r_x}{r_y}}$$

Values of the ratio r_x/r_y and other properties useful in the design of W-shape compression members are listed at the bottom of Table 4–1. The variables P_{wo} and P_{wi} can be used in the calculation of the web local yielding available strength (AISC Specification Equation J10-2) for the column as follows:

LRFD	ASD
$\phi R_n = P_{wo} + P_{wi}N$	$R_n/\Omega = P_{wo} + P_{wi}N$

The variable P_{wb} can be used in the calculation of the available web compression buckling strength (AISC Specification Equation J10-8) for the column as follows:

LRFD	ASD
$\phi R_n = P_{wb}$	$R_n/\Omega = P_{wb}$

The variable P_{fb} can be used in the calculation of the available flange local bending strength (AISC Specification Equation J10-1) for the column as follows:

LRFD	ASD
$\phi R_n = P_{fb}$	$R_n/\Omega = P_{fb}$

Table 4–2. HP-Shapes in Axial Compression

Table 4–2 is similar to Table 4–1, except it covers HP-shapes with $F_y = 50$ ksi (ASTM A572 grade 50).

Table 4–3. Rectangular HSS in Axial Compression

Available strengths in axial compression are given for rectangular HSS with $F_y = 46$ ksi (ASTM A500 grade B). The tabulated values are given for the effective length with respect to the Y-Y axis $(KL)_y$. However, the effective length with respect to the X-X axis $(KL)_x$ must also be investigated. To determine the available strength in axial compression, the table should be entered at the larger of $(KL)_y$ and $(KL)_{y\,eq}$, where

$$(KL)_{y\,eq} = \frac{(KL)_x}{\frac{r_x}{r_y}}$$

Values of the ratio r_x/r_y and other properties useful in the design of rectangular HSS compression members are listed at the bottom of Table 4–3.

Table 4–4. Square HSS in Axial Compression

Table 4–4 is similar to Table 4–3, except that it covers square HSS.

Table 4–5. Round HSS in Axial Compression

Available strengths in axial compression are given for round HSS with $F_y = 42$ ksi (ASTM A500 grade B). To determine the available strength in axial compression, the table should be entered at KL. Other properties useful in the design of compression members are listed at the bottom of the available column strength tables.

Table 4–6. Pipe in Axial Compression

Table 4–6 is similar to Table 4-5, except it covers pipe with $F_y = 35$ ksi (ASTM A53 grade B).

Table 4–7. WT-Shapes in Axial Compression

Available strengths in axial compression are given for WT-shapes with $F_y = 50$ ksi (ASTM A992). Separate tabulated values are given for the effective lengths with respect to the

X-X and Y-Y axes, $(KL)_x$ and $(KL)_y$, respectively. Other properties useful in the design of WT-shape compression members are listed at the bottom of Table 4-7.

Table 4–8. Equal-Leg Double Angles in Axial Compression

Available strengths in axial compression are given for equal-leg double angles with $F_y = 36$ ksi (ASTM A36), assuming 3/8-in. separation between the angles. These values can be used conservatively when a larger separation is provided. Alternatively, the value of $(KL)_y$ can be multiplied by the ratio of (r_y for a 3/8-in. separation) to (r_y for the actual separation).

Separate tabulated values are given for the effective lengths with respect to the X-X and Y-Y axes, $(KL)_x$ and $(KL)_y$, respectively. For buckling about the X-X axis, the available strength is not affected by the number of intermediate connectors. However, for buckling about the Y-Y axis, the effects of shear deformations of the intermediate connectors must be considered. The tabulated values for $(KL)_y$ have been adjusted for the shear deformations in accordance with AISC Specification equation E6-2, which is applicable to welded and pretensioned bolted intermediate shear connectors. The number of intermediate connectors, n, is given in the table and the line of demarcation between the required connector values is dashed. Intermediate connectors are selected such that the available compression buckling strength about the Y-Y axis is equal to or greater than 90 percent of that for compression buckling of the two angles as a unit. If fewer connectors or snug-tightened bolted intermediate connectors are used, the available strength must be recalculated per AISC Specification Section E6. Per AISC Specification Section E6.2, the slenderness of the individual components of the built-up member based upon the distance between intermediate connectors, a, must not exceed three-quarters of the controlling slenderness of the overall built-up compression member.

Other properties useful in the design of double-angle compression members are listed at the bottom of Table 4–8.

Table 4–9. LLBB Double Angles in Axial Compression

Table 4–9 is the same as Table 4–8, except that it provides available strengths in axial compression for double angles with long legs back to back.

Table 4–10. SLBB Double Angles in Axial Compression

Table 4–10 is the same as Table 4–8, except that it provides available strengths in axial compression for double angles with short legs back to back.

Table 4–11. Concentrically Loaded Single Angles in Axial Compression

Available strengths in axial compression are given for single angles, loaded through the centroid of the cross-section, with $F_y = 36$ ksi (ASTM A36) based upon the effective length with respect to the Z-Z axis $(KL)_z$. Single angles may be assumed to be loaded through the centroid when the requirements of AISC Specification Section E5 are met, as in these cases the eccentricity is accounted for and the slenderness is reduced by the restraining effects of the support at both ends of the member.

Table 4–12. Eccentrically Loaded Single Angles in Axial Compression

Available strengths in axial compression are given for single angles with $F_y = 36$ ksi (ASTM A36). These tables present a lower-bound available axial strength for eccentrically loaded single angles without consideration of end restraint (Sakla, 2001) and may be used in the design of single angle compression members when the requirements of Specification Section E5 can not be met.

In the development of this table, *KL* is assumed to be the same on all axes (r_x, r_y, r_z, and r_w). To determine the available strength in axial compression, the table should be entered at the largest effective length between brace points. These tables consider combined biaxial bending about the principal axes with axial compression. The long leg of the angle is assumed to be attached to a gusset with a thickness of 1.5*t*. The tabulated values assume a load placed at the center of the gusset plate, at a distance of 0.75*t* from the long leg of the angle.

COMPOSITE COMPRESSION—MEMBER SELECTION TABLES

Table 4–13. Rectangular HSS Filled with 4-ksi Normal-Weight Concrete in Axial Compression

Available strengths in axial compression are given for rectangular HSS with F_y = 46 ksi (ASTM A500 grade B) filled with 4-ksi normal-weight concrete. The tabulated values are given for the effective length with respect to the Y-Y axis $(KL)_y$. However, the effective length with respect to the X-X axis $(KL)_x$ must also be investigated. To determine the available strength in axial compression, the table should be entered at the larger of $(KL)_y$ and $(KL)_{y\,eq}$, where

$$(KL)_{y\,eq} = \frac{(KL)_x}{\dfrac{r_{mx}}{r_{my}}}$$

Values of the ratio r_{mx}/r_{my} and other properties useful in the design of composite HSS compression members are listed at the bottom of Table 4–13. The variables r_{mx} and r_{my} are the radii of gyration for the composite cross-section. The ratio r_{mx}/r_{my} is determined as

$$\frac{r_{mx}}{r_{my}} = \sqrt{\frac{P_{ex}(K_xL_x)^2}{P_{ey}(K_yL_y)^2}}$$

Table 4–14. Square HSS Filled with 4-ksi Normal-Weight Concrete in Axial Compression

Table 4–14 is the same as Table 4–13, except that it provides available strengths in axial compression for square HSS filled with 4-ksi normal-weight concrete.

Table 4–15. Rectangular HSS Filled with 5-ksi Normal-Weight Concrete in Axial Compression

Table 4–15 is the same as Table 4–13, except that it provides available strengths in axial compression for rectangular HSS filled with 5-ksi normal-weight concrete.

Table 4–16. Square HSS Filled with 5-ksi Normal-Weight Concrete in Axial Compression

Table 4–16 is the same as Table 4–13, except that it provides available strengths in axial compression for square HSS filled with 5-ksi normal-weight concrete.

Table 4–17. Round HSS Filled with 4-ksi Normal-Weight Concrete in Axial Compression

Available strengths in axial compression are given for round HSS with F_y = 42 ksi (ASTM A500 grade B) filled with 4-ksi normal-weight concrete. To determine the available strength in axial compression, the table should be entered at the largest effective length, *KL*. Other properties useful in the design of compression members are listed at the bottom of the column available strength tables.

Table 4–18. Round HSS Filled with 5-ksi Normal-Weight Concrete in Axial Compression

Table 4–18 is the same as Table 4–17, except that it provides available strengths in axial compression for round HSS filled with 5-ksi normal-weight concrete.

Table 4–19. Pipe Filled with 4-ksi Normal-Weight Concrete in Axial Compression

Available strengths in axial compression are given for pipe with F_y = 35 ksi (ASTM A53 grade B) filled with 4-ksi normal-weight concrete. To determine the available strength in axial compression, the table should be entered at the largest effective length, *KL*. Other properties useful in the design of compression members are listed at the bottom of the column available strength tables.

Table 4–20. Pipe Filled with 5-ksi Normal-Weight Concrete in Axial Compression

Table 4–21 is the same as Table 4–20, except that it provides available strengths in axial compression for pipe filled with 5-ksi normal-weight concrete.

Table 4–21. Stiffness Reduction Factor τ_a

When column buckling occurs in the inelastic range, the use of the alignment charts in Chapter C of the Commentary usually gives conservative results. For more accurate solutions, inelastic *K*-factors can be determined from the alignment chart by using τ_a times the elastic modulus, E_c, of the columns in the equation for *G*. The stiffness reduction factor, τ_a, is the ratio of the tangent modulus, E_T , to the elastic modulus, *E*. Values are tabulated for steels with F_y = 35 ksi, 36 ksi, 42 ksi, 46 ksi, and 50 ksi.

Table 4–22. Available Critical Stress for Compression Members

Table 4–22 provides the available critical stress for various ratios of *Kl/r*, for materials with a minimum specified yield strength of 35 ksi, 36 ksi, 42 ksi, 46 ksi, and 50 ksi.

PART 4 REFERENCES

Galambos, T.V., 1998, *Guide to Stability Design Criteria for Metal Structures*, Fifth Edition, John Wiley and Sons, Inc., New York, NY.

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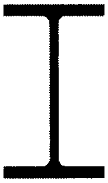


W14

Table 4–1

Available Strength in Axial Compression, kips

W Shapes

$F_y = 50$ ksi

Shape		W14×											
Wt/ft		730^h		665^h		605^h		550^h		500^h		455^h	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	6440	9670	5870	8820	5330	8010	4850	7290	4400	6610	4010	6030
	11	6070	9130	5530	8310	5010	7530	4550	6840	4120	6200	3750	5640
	12	6010	9030	5470	8220	4950	7440	4500	6760	4070	6120	3710	5570
	13	5940	8920	5400	8110	4890	7350	4440	6670	4020	6040	3660	5500
	14	5860	8810	5330	8010	4820	7250	4380	6580	3960	5950	3600	5420
	15	5780	8690	5250	7890	4750	7140	4310	6480	3900	5860	3550	5330
	16	5690	8560	5170	7770	4680	7030	4240	6380	3840	5770	3490	5240
	17	5610	8430	5090	7650	4600	6920	4170	6270	3770	5660	3420	5150
	18	5510	8290	5000	7520	4520	6790	4100	6160	3700	5560	3360	5050
	19	5420	8140	4910	7380	4440	6670	4020	6040	3630	5450	3290	4950
	20	5320	7990	4820	7240	4350	6540	3940	5920	3550	5340	3220	4840
	22	5110	7670	4620	6950	4170	6260	3770	5660	3390	5100	3080	4620
	24	4890	7340	4420	6640	3980	5980	3590	5400	3230	4860	2920	4400
	26	4660	7000	4200	6320	3780	5680	3410	5120	3060	4600	2770	4160
	28	4420	6650	3990	5990	3580	5380	3220	4840	2890	4340	2610	3920
	30	4180	6290	3760	5660	3370	5070	3030	4560	2720	4080	2450	3680
	32	3940	5930	3540	5320	3170	4760	2840	4270	2540	3820	2290	3440
	34	3700	5560	3320	4990	2960	4450	2650	3990	2370	3560	2130	3200
	36	3460	5200	3100	4650	2760	4140	2460	3700	2200	3300	1970	2960
	38	3220	4850	2880	4330	2560	3840	2280	3430	2030	3050	1820	2730
	40	2990	4500	2670	4010	2360	3550	2100	3160	1870	2800	1670	2510
	42	2770	4160	2460	3690	2170	3270	1930	2900	1710	2570	1520	2290
	44	2550	3830	2260	3390	1990	2990	1760	2650	1560	2340	1390	2080
	46	2330	3510	2060	3100	1820	2730	1610	2420	1420	2140	1270	1910
	48	2140	3220	1900	2850	1670	2510	1480	2220	1310	1960	1160	1750
	50	1970	2970	1750	2630	1540	2310	1360	2050	1200	1810	1070	1610
Properties													
P_{wo} (kips)		2820	4230	2410	3620	2060	3090	1750	2630	1500	2240	1280	1920
P_{wi} (kips/in.)		102	154	94.3	142	86.5	130	79.3	119	73.0	110	67.2	101
P_{wb} (kips)		43900	66000	34400	51700	26500	39900	20500	30700	15900	24000	12400	18700
P_{fb} (kips)		4510	6780	3820	5750	3240	4870	2730	4100	2290	3450	1930	2900
L_p (ft)		16.6		16.3		16.1		15.9		15.6		15.5	
L_r (ft)		275		253		232		213		196		179	
A_g (in.^{2})		215		196		178		162		147		134	
I_x (in.^{4})		14300		12400		10800		9430		8210		7190	
I_y (in.^{4})		4720		4170		3680		3250		2880		2560	
r_y (in.)		4.69		4.62		4.55		4.49		4.43		4.38	
Ratio r_x/r_y		1.74		1.73		1.71		1.70		1.69		1.67	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		409000		355000		309000		270000		235000		206000	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		135000		119000		105000		93000		82400		73300	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 4-1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W14

Shape		W14×											
Wt/ft		426[h]		398[h]		370[h]		342[h]		311[h]		283[h]	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	3740	5620	3500	5260	3260	4900	3020	4540	2740	4110	2490	3750
	11	3500	5260	3270	4920	3040	4570	2820	4230	2550	3830	2320	3480
	12	3450	5190	3230	4850	3000	4510	2780	4180	2510	3780	2280	3430
	13	3410	5120	3180	4780	2960	4450	2740	4120	2470	3720	2250	3380
	14	3350	5040	3130	4710	2910	4380	2700	4050	2440	3660	2210	3330
	15	3300	4960	3080	4630	2870	4310	2650	3980	2390	3600	2180	3270
	16	3240	4870	3030	4550	2810	4230	2600	3910	2350	3530	2130	3210
	17	3180	4790	2970	4470	2760	4150	2550	3840	2300	3460	2090	3150
	18	3120	4690	2920	4380	2710	4070	2500	3760	2260	3390	2050	3080
	19	3060	4600	2850	4290	2650	3980	2450	3680	2210	3320	2000	3010
	20	2990	4500	2790	4200	2590	3890	2390	3600	2160	3240	1960	2940
	22	2860	4290	2660	4000	2470	3710	2280	3420	2050	3080	1860	2790
	24	2710	4080	2530	3800	2340	3520	2160	3240	1940	2920	1760	2640
	26	2560	3850	2390	3590	2210	3320	2040	3060	1830	2750	1660	2490
	28	2410	3630	2250	3380	2080	3120	1910	2870	1710	2580	1550	2330
	30	2260	3400	2100	3160	1940	2920	1790	2680	1600	2400	1450	2170
	32	2110	3170	1960	2950	1810	2720	1660	2500	1490	2230	1340	2020
	34	1960	2950	1820	2730	1670	2520	1540	2310	1370	2060	1240	1860
	36	1810	2730	1680	2530	1540	2320	1420	2130	1260	1900	1140	1710
	38	1670	2510	1550	2320	1420	2130	1300	1950	1160	1740	1040	1560
	40	1530	2300	1410	2130	1300	1950	1180	1780	1050	1580	944	1420
	42	1390	2090	1290	1930	1180	1770	1070	1610	954	1430	857	1290
	44	1270	1910	1170	1760	1070	1610	979	1470	870	1310	781	1170
	46	1160	1750	1070	1610	980	1470	896	1350	796	1200	714	1070
	48	1070	1600	985	1480	900	1350	823	1240	731	1100	656	986
	50	983	1480	907	1360	830	1250	758	1140	673	1010	604	909
Properties													
P_{wo} (kips)		1140	1700	1020	1520	899	1350	787	1180	672	1010	574	860
P_{wi} (kips/in.)		62.5	93.8	59.0	88.5	55.2	82.8	51.3	77.0	47.0	70.5	43.0	64.5
P_{wb} (kips)		10000	15000	8410	12600	6880	10300	5540	8330	4250	6390	3260	4900
P_{fb} (kips)		1720	2590	1510	2280	1320	1990	1140	1720	956	1440	802	1210
L_p (ft)		15.3		15.2		15.1		15.0		14.8		14.7	
L_r (ft)		169		158		148		137		125		114	
A_g (in.^{2})		125		117		109		101		91.4		83.3	
I_x (in.^{4})		6600		6000		5440		4900		4330		3840	
I_y (in.^{4})		2360		2170		1990		1810		1610		1440	
r_y (in.)		4.34		4.31		4.27		4.24		4.20		4.17	
Ratio r_x/r_y		1.67		1.66		1.66		1.65		1.64		1.63	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		189000		172000		156000		140000		124000		110000	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		67500		62100		57000		51800		46100		41200	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W14

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

Shape		W14×											
Wt/ft		257		233		211		193		176		159	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	2260	3400	2050	3080	1850	2790	1700	2550	1550	2330	1400	2100
	6	2210	3330	2000	3010	1810	2720	1660	2500	1510	2280	1370	2050
	7	2200	3300	1990	2990	1800	2700	1650	2480	1500	2260	1350	2040
	8	2180	3270	1970	2960	1780	2680	1630	2450	1490	2230	1340	2020
	9	2150	3230	1950	2930	1760	2650	1610	2430	1470	2210	1330	1990
	10	2130	3200	1920	2890	1740	2620	1590	2400	1450	2180	1310	1970
	11	2100	3160	1900	2860	1720	2580	1570	2360	1430	2150	1290	1940
	12	2070	3110	1870	2810	1690	2540	1550	2330	1410	2120	1270	1910
	13	2040	3060	1840	2770	1670	2500	1520	2290	1390	2090	1250	1880
	14	2000	3010	1810	2720	1640	2460	1500	2250	1360	2050	1230	1850
	15	1970	2960	1780	2680	1610	2420	1470	2210	1340	2010	1210	1810
	16	1930	2900	1750	2620	1580	2370	1440	2170	1310	1970	1180	1780
	17	1890	2850	1710	2570	1540	2320	1410	2120	1280	1930	1160	1740
	18	1850	2780	1670	2510	1510	2270	1380	2070	1250	1890	1130	1700
	19	1810	2720	1630	2460	1470	2220	1350	2030	1230	1840	1100	1660
	20	1770	2660	1600	2400	1440	2160	1310	1980	1190	1800	1080	1620
	22	1680	2520	1510	2270	1360	2050	1250	1870	1130	1700	1020	1530
	24	1590	2380	1430	2150	1290	1930	1170	1760	1060	1600	958	1440
	26	1490	2240	1340	2020	1210	1810	1100	1660	998	1500	897	1350
	28	1390	2100	1250	1890	1130	1690	1030	1540	930	1400	835	1260
	30	1300	1950	1170	1750	1050	1570	954	1430	862	1300	774	1160
	32	1200	1810	1080	1620	967	1450	881	1320	795	1200	713	1070
	34	1110	1670	994	1490	890	1340	809	1220	730	1100	654	983
	36	1020	1530	910	1370	814	1220	740	1110	666	1000	596	896
	38	928	1390	830	1250	741	1110	673	1010	605	909	541	812
	40	841	1260	751	1130	669	1010	607	913	546	820	488	733
Properties													
P_{wo} (kips)		487	731	413	620	352	529	302	453	264	396	222	333
P_{wi} (kips/in.)		39.2	58.8	35.7	53.5	32.7	49.0	29.7	44.5	27.7	41.5	24.8	37.3
P_{wb} (kips)		2460	3700	1860	2790	1430	2150	1070	1610	868	1300	627	943
P_{fb} (kips)		668	1000	554	832	455	684	388	583	321	483	265	398
L_p (ft)		14.6		14.5		14.4		14.3		14.2		14.1	
L_r (ft)		104		94.9		86.4		79.7		73.2		66.7	
A_g (in.^{2})		75.6		68.5		62.0		56.8		51.8		46.7	
I_x (in.^{4})		3400		3010		2660		2400		2140		1900	
I_y (in.^{4})		1290		1150		1030		931		838		748	
r_y (in.)		4.13		4.10		4.07		4.05		4.02		4.00	
Ratio r_x/r_y		1.62		1.62		1.61		1.60		1.60		1.60	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		97300		86200		76100		68700		61300		54400	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		36900		32900		29500		26600		24000		21400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 50 ksi

Table 4–1 (continued)

Available Strength in Axial Compression, kips

W Shapes

W14

Shape		W14×											
Wt/ft		145		132		120		109		99		90	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	1280	1920	1160	1740	1060	1590	959	1440	872	1310	792	1190
	6	1250	1870	1130	1700	1030	1550	934	1400	849	1280	771	1160
	7	1240	1860	1120	1680	1020	1530	924	1390	840	1260	763	1150
	8	1220	1840	1110	1660	1010	1510	914	1370	831	1250	754	1130
	9	1210	1820	1090	1640	995	1500	902	1360	820	1230	745	1120
	10	1200	1800	1080	1620	981	1470	889	1340	808	1210	734	1100
	11	1180	1770	1060	1590	965	1450	875	1320	795	1200	722	1090
	12	1160	1740	1040	1570	949	1430	860	1290	781	1170	709	1070
	13	1140	1720	1020	1540	931	1400	844	1270	767	1150	696	1050
	14	1120	1690	1000	1510	912	1370	827	1240	751	1130	682	1020
	15	1100	1650	982	1480	893	1340	809	1220	734	1100	667	1000
	16	1080	1620	959	1440	872	1310	790	1190	717	1080	651	978
	17	1050	1580	936	1410	851	1280	771	1160	699	1050	635	954
	18	1030	1550	912	1370	829	1250	751	1130	681	1020	618	928
	19	1000	1510	887	1330	806	1210	730	1100	662	995	600	902
	20	979	1470	862	1300	783	1180	709	1070	642	966	583	876
	22	926	1390	809	1220	735	1100	665	1000	602	906	546	821
	24	871	1310	756	1140	685	1030	620	932	562	844	509	765
	26	815	1230	702	1050	636	956	575	864	520	782	471	708
	28	759	1140	647	973	586	881	530	797	479	720	434	652
	30	702	1060	594	892	537	807	485	730	438	659	397	596
	32	647	972	541	814	489	735	442	664	399	599	361	542
	34	592	890	491	738	443	666	400	601	360	542	326	490
	36	540	811	441	663	398	598	359	540	323	486	292	439
	38	489	734	396	595	357	537	322	484	290	436	262	394
	40	441	663	358	537	322	484	291	437	262	393	236	355
Properties													
P_{wo} (kips)		191	287	175	263	151	227	128	191	111	167	95.9	144
P_{wi} (kips/in.)		22.7	34.0	21.5	32.3	19.7	29.5	17.5	26.3	16.2	24.3	14.7	22.0
P_{wb} (kips)		477	717	407	612	312	468	220	330	173	260	129	194
P_{fb} (kips)		222	334	199	298	165	249	138	208	114	171	94.3	142
L_p (ft)		14.1		13.3		13.2		13.2		13.5		15.2	
L_r (ft)		61.7		56.0		52.0		48.4		45.3		42.6	
A_g (in.^{2})		42.7		38.8		35.3		32.0		29.1		26.5	
I_x (in.^{4})		1710		1530		1380		1240		1110		999	
I_y (in.^{4})		677		548		495		447		402		362	
r_y (in.)		3.98		3.76		3.74		3.73		3.71		3.70	
Ratio r_x/r_y		1.59		1.67		1.67		1.67		1.66		1.66	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		48900		43800		39500		35500		31800		28600	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		19400		15700		14200		12800		11500		10400	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

W14

Table 4-1 (continued)

Available Strength in Axial Compression, kips

W Shapes

F_y = 50 ksi

Shape		W14×													
Wt/ft		82		74		68		61		53		48		43^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	720	1080	652	980	598	899	536	806	467	702	423	636	374	562
	6	677	1020	613	922	562	844	504	757	421	633	382	573	340	511
	7	662	995	600	901	549	826	492	740	406	610	368	552	327	491
	8	645	970	584	878	535	804	480	721	389	585	352	529	313	470
	9	627	942	568	853	520	781	465	700	371	557	335	504	298	447
	10	607	912	549	826	503	755	450	677	351	528	317	477	281	423
	11	585	880	530	797	485	728	434	652	331	497	299	449	264	397
	12	563	846	510	766	466	700	417	626	310	465	279	420	247	371
	13	539	810	488	734	446	670	399	599	288	433	260	391	230	345
	14	515	774	466	701	425	639	380	572	267	401	240	361	212	319
	15	490	736	444	667	404	608	362	543	246	369	221	332	195	293
	16	465	698	421	632	383	576	342	515	225	338	202	304	178	267
	17	439	660	398	598	362	544	323	486	205	308	184	276	161	242
	18	413	621	374	563	340	512	304	457	185	278	166	250	145	218
	19	388	583	351	528	319	480	285	428	166	250	149	224	130	196
	20	363	546	329	494	298	448	266	400	150	226	135	202	118	177
	22	314	473	285	428	258	387	230	345	124	186	111	167	97.2	146
	24	268	403	243	365	219	329	195	293	104	157	93.5	140	81.7	123
	26	228	343	207	311	187	281	166	250	88.8	133	79.6	120	69.6	105
	28	197	296	178	268	161	242	143	215	76.6	115	68.7	103	60.0	90.2
	30	172	258	155	234	140	211	125	187	66.7	100	59.8	89.9	52.3	78.6
	32	151	227	137	205	123	185	110	165	58.6	88.1				
	34	134	201	121	182	109	164	97.1	146						
	36	119	179	108	162	97.4	146	86.6	130						
	38	107	161	96.8	146	87.4	131	77.7	117						
	40	96.5	145	87.4	131	78.9	119	70.2	105						
Properties															
P_{wo} (kips)		123	184	103	155	90.7	136	77.3	116	77.1	116	67.2	101	57.0	85.5
P_{wi} (kips/in.)		17.0	25.5	15.0	22.5	13.8	20.8	12.5	18.8	12.3	18.5	11.3	17.0	10.2	15.3
P_{wb} (kips)		201	302	138	208	108	163	79.9	120	76.8	115	59.6	89.5	43.0	64.6
P_{fb} (kips)		137	206	115	173	97.0	146	77.8	117	81.5	123	66.2	99.6	52.6	79.0
L_p (ft)		8.76		8.76		8.69		8.65		6.78		6.75		6.68	
L_r (ft)		33.1		31.0		29.3		27.5		22.2		21.1		20.0	
A_g (in.^{2})		24.0		21.8		20.0		17.9		15.6		14.1		12.6	
I_x (in.^{4})		881		795		722		640		541		484		428	
I_y (in.^{4})		148		134		121		107		57.7		51.4		45.2	
r_y (in.)		2.48		2.48		2.46		2.45		1.92		1.91		1.89	
Ratio r_x/r_y		2.44		2.44		2.44		2.44		3.07		3.06		3.08	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		25200		22800		20700		18300		15500		13900		12300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		4240		3840		3460		3060		1650		1470		1290	

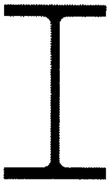
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes



W12

Shape		W12×											
Wt/ft		336^h		305^h		279^h		252^h		230^h		210	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	2960	4440	2680	4030	2450	3690	2220	3330	2030	3050	1850	2780
	6	2870	4310	2600	3910	2370	3570	2140	3220	1960	2940	1790	2680
	7	2830	4260	2570	3860	2340	3520	2120	3180	1930	2910	1760	2650
	8	2800	4200	2530	3810	2310	3470	2090	3140	1910	2870	1740	2610
	9	2760	4140	2500	3750	2280	3420	2050	3090	1880	2820	1710	2570
	10	2710	4070	2450	3690	2240	3360	2020	3030	1840	2770	1680	2520
	11	2660	4000	2410	3620	2190	3300	1980	2970	1800	2710	1640	2470
	12	2610	3920	2360	3540	2150	3230	1940	2910	1770	2650	1610	2410
	13	2550	3830	2310	3460	2100	3150	1890	2840	1720	2590	1570	2360
	14	2490	3740	2250	3380	2050	3080	1840	2770	1680	2520	1530	2290
	15	2430	3650	2190	3290	1990	3000	1790	2690	1630	2450	1480	2230
	16	2360	3550	2130	3200	1940	2910	1740	2620	1590	2380	1440	2160
	17	2300	3450	2070	3110	1880	2820	1690	2540	1540	2310	1390	2100
	18	2230	3350	2000	3010	1820	2730	1630	2450	1480	2230	1350	2020
	19	2160	3240	1940	2910	1760	2640	1580	2370	1430	2150	1300	1950
	20	2080	3130	1870	2810	1700	2550	1520	2280	1380	2070	1250	1880
	22	1940	2910	1740	2610	1570	2360	1400	2110	1270	1910	1150	1730
	24	1790	2690	1600	2400	1440	2170	1290	1930	1170	1750	1050	1580
	26	1640	2460	1460	2190	1320	1980	1170	1760	1060	1590	955	1430
	28	1490	2240	1330	1990	1190	1790	1060	1590	954	1430	859	1290
	30	1350	2020	1190	1790	1070	1610	948	1430	854	1280	767	1150
	32	1210	1820	1070	1600	954	1430	842	1270	756	1140	678	1020
	34	1070	1610	947	1420	845	1270	746	1120	670	1010	600	902
	36	958	1440	844	1270	754	1130	665	1000	597	898	535	805
	38	860	1290	758	1140	676	1020	597	897	536	806	480	722
	40	776	1170	684	1030	610	918	539	810	484	727	434	652
Properties													
P_{wo} (kips)		1050	1580	895	1340	782	1170	662	993	571	857	491	737
P_{wi} (kips/in.)		59.2	88.8	54.2	81.3	51.0	76.5	46.5	69.8	42.8	64.3	39.3	59.0
P_{wb} (kips)		9960	15000	7640	11500	6380	9590	4840	7270	3780	5680	2930	4400
P_{fb} (kips)		1630	2460	1370	2060	1140	1720	947	1420	802	1210	676	1020
L_p (ft)		12.3		12.1		11.9		11.8		11.7		11.6	
L_r (ft)		150		137		126		114		105		96.0	
A_g (in.^{2})		98.8		89.6		81.9		74.0		67.7		61.8	
I_x (in.^{4})		4060		3550		3110		2720		2420		2140	
I_y (in.^{4})		1190		1050		937		828		742		664	
r_y (in.)		3.47		3.42		3.38		3.34		3.31		3.28	
Ratio r_x/r_y		1.85		1.84		1.82		1.81		1.80		1.80	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		116000		102000		89000		77900		69300		61300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		34100		30100		26800		23700		21200		19000	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W12

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

Shape		W12×											
Wt/ft		190		170		152		136		120		106	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	1670	2510	1500	2250	1340	2010	1200	1800	1060	1590	933	1400
	6	1610	2420	1440	2170	1290	1940	1150	1730	1020	1530	897	1350
	7	1590	2390	1420	2140	1270	1910	1140	1710	1000	1510	884	1330
	8	1570	2360	1400	2110	1250	1880	1120	1680	986	1480	870	1310
	9	1540	2320	1380	2070	1230	1850	1100	1650	968	1460	854	1280
	10	1510	2270	1350	2030	1210	1820	1080	1620	949	1430	837	1260
	11	1480	2230	1320	1990	1180	1780	1050	1580	927	1390	818	1230
	12	1450	2180	1290	1940	1150	1730	1030	1540	905	1360	798	1200
	13	1410	2120	1260	1900	1120	1690	1000	1500	881	1320	776	1170
	14	1380	2070	1230	1840	1090	1640	972	1460	856	1290	754	1130
	15	1340	2010	1190	1790	1060	1600	943	1420	829	1250	730	1100
	16	1300	1950	1150	1730	1030	1540	913	1370	802	1210	706	1060
	17	1250	1880	1120	1680	993	1490	881	1320	774	1160	681	1020
	18	1210	1820	1080	1620	958	1440	849	1280	746	1120	656	985
	19	1170	1750	1040	1560	922	1390	817	1230	717	1080	630	946
	20	1120	1690	997	1500	886	1330	784	1180	687	1030	604	907
	22	1030	1550	916	1380	812	1220	718	1080	628	944	551	828
	24	941	1420	834	1250	738	1110	651	979	569	855	498	749
	26	852	1280	754	1130	666	1000	586	881	511	768	447	672
	28	765	1150	675	1010	595	895	523	786	455	684	397	597
	30	682	1020	600	902	528	793	462	695	401	602	350	525
	32	601	904	528	794	464	698	406	611	352	530	307	462
	34	533	800	468	704	411	618	360	541	312	469	272	409
	36	475	714	418	628	367	551	321	483	278	418	243	365
	38	426	641	375	563	329	495	288	433	250	376	218	327
	40	385	578	338	508	297	446	260	391	225	339	197	295
Properties													
P_{wo} (kips)		412	618	345	518	290	435	243	365	202	302	161	242
P_{wi} (kips/in.)		35.3	53.0	32.0	48.0	29.0	43.5	26.3	39.5	23.7	35.5	20.3	30.5
P_{wb} (kips)		2120	3190	1580	2370	1170	1760	878	1320	638	958	404	608
P_{fb} (kips)		563	847	455	684	367	551	292	439	228	343	183	276
L_p (ft)		11.5		11.4		11.3		11.2		11.1		11.0	
L_r (ft)		87.3		78.5		70.6		63.3		56.5		50.7	
A_g (in.^{2})		55.8		50.0		44.7		39.9		35.3		31.2	
I_x (in.^{4})		1890		1650		1430		1240		1070		933	
I_y (in.^{4})		589		517		454		398		345		301	
r_y (in.)		3.25		3.22		3.19		3.16		3.13		3.11	
Ratio r_x/r_y		1.79		1.78		1.77		1.77		1.76		1.76	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		54100		47200		40900		35500		30600		26700	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		16900		14800		13000		11400		9870		8620	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 50 ksi

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

W12

Shape		W12×									
Wt/ft		96		87		79		72		65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	844	1270	766	1150	694	1040	633	951	571	859
	6	811	1220	735	1110	667	1000	607	913	548	824
	7	800	1200	725	1090	657	987	598	899	540	811
	8	787	1180	713	1070	646	971	588	884	531	798
	9	772	1160	699	1050	634	952	577	867	520	782
	10	756	1140	685	1030	620	932	565	849	509	765
	11	739	1110	669	1010	606	910	551	828	497	747
	12	720	1080	652	980	590	887	537	807	484	727
	13	701	1050	634	953	573	862	522	784	470	706
	14	680	1020	615	924	556	836	506	761	456	685
	15	659	990	595	895	538	809	490	736	441	662
	16	637	957	575	864	520	781	473	710	425	639
	17	614	923	554	833	501	752	455	684	409	615
	18	591	888	533	801	481	723	437	657	393	591
	19	567	852	511	769	461	694	419	630	377	566
	20	543	816	490	736	442	664	401	603	360	541
	22	495	744	446	670	402	603	365	548	327	491
	24	447	672	402	605	362	544	328	493	294	442
	26	401	602	360	541	323	486	293	440	262	393
	28	356	534	319	479	286	430	259	389	231	347
	30	312	469	279	420	250	376	226	340	202	303
	32	274	412	246	369	220	331	199	299	177	267
	34	243	365	218	327	195	293	176	265	157	236
	36	217	326	194	292	174	261	157	236	140	211
	38	195	292	174	262	156	234	141	212	126	189
	40	176	264	157	236	141	212	127	191	114	171

Properties

	96		87		79		72		65	
P_{wo} (kips)	137	206	121	181	104	157	90.9	136	78.2	117
P_{wi} (kips/in.)	18.3	27.5	17.2	25.8	15.7	23.5	14.3	21.5	13.0	19.5
P_{wb} (kips)	296	445	243	366	185	278	142	213	106	159
P_{fb} (kips)	152	228	123	185	101	152	84.0	126	68.5	103
L_p (ft)	10.9		10.8		10.8		10.7		11.9	
L_r (ft)	46.6		43.0		39.9		37.4		35.1	
A_g (in.^{2})	28.2		25.6		23.2		21.1		19.1	
I_x (in.^{4})	833		740		662		597		533	
I_y (in.^{4})	270		241		216		195		174	
r_y (in.)	3.09		3.07		3.05		3.04		3.02	
Ratio r_x/r_y	1.76		1.75		1.75		1.75		1.75	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	23800		21200		18900		17100		15300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	7730		6900		6180		5580		4980	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

W12

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

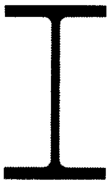
F_y = 50 ksi

Shape		**W12×**									
Wt/ft		**58**		**53**		**50**		**45**		**40**	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y	**0**	510	767	466	701	437	657	393	590	350	526
	6	481	722	438	659	396	595	356	534	316	475
	7	470	707	429	644	382	574	343	516	305	458
	8	459	689	418	628	367	551	329	495	292	439
	9	446	670	406	610	350	526	314	472	279	419
	10	432	649	393	590	332	499	298	448	264	397
	11	417	627	379	569	314	471	281	422	249	375
	12	401	603	364	547	294	443	264	396	234	351
	13	385	578	349	525	275	413	246	370	218	328
	14	368	553	333	501	255	384	228	343	202	304
	15	350	527	317	477	236	354	211	317	186	280
	16	333	500	301	452	217	326	193	291	171	257
	17	315	473	284	427	198	297	176	265	156	234
	18	297	446	268	402	180	270	160	241	141	212
	19	279	420	251	378	162	244	144	217	127	191
	20	262	393	235	353	146	220	130	196	115	172
	22	227	342	204	306	121	182	108	162	94.8	142
	24	195	293	174	261	102	153	90.4	136	79.6	120
	26	166	249	148	222	86.6	130	77.0	116	67.9	102
	28	143	215	127	192	74.6	112	66.4	99.8	58.5	88.0
	30	125	187	111	167	65.0	97.7	57.9	87.0	51.0	76.6
	32	109	165	97.6	147	57.1	85.9	50.9	76.4	44.8	67.3
	34	97.0	146	86.5	130						
	36	86.5	130	77.1	116						
	38	77.6	117	69.2	104						
	40	70.1	105	62.5	93.9						
Properties											
P_{wo} (kips)		74.4	112	67.6	101	70.3	105	60.0	90.0	49.9	74.9
P_{wi} (kips/in.)		12.0	18.0	11.5	17.3	12.3	18.5	11.2	16.8	9.83	14.8
P_{wb} (kips)		83.2	125	73.2	110	88.5	133	65.7	98.7	44.8	67.4
P_{fb} (kips)		76.6	115	61.9	93.0	76.6	115	61.9	93.0	49.6	74.6
L_p (ft)		8.87		8.76		6.92		6.89		6.85	
L_r (ft)		29.9		28.2		23.9		22.4		21.1	
A_g (in.^{2})		17.0		15.6		14.6		13.1		11.7	
I_x (in.^{4})		475		425		391		348		307	
I_y (in.^{4})		107		95.8		56.3		50.0		44.1	
r_y (in.)		2.51		2.48		1.96		1.95		1.94	
Ratio r_x/r_y		2.10		2.11		2.64		2.64		2.64	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		13600		12200		11200		9960		8790	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		3060		2740		1610		1430		1260	

ASD	**LRFD**	Note: Heavy line indicates *Kl/r* equal to or greater than 200.
$\Omega_c = 1.67$	$\phi_c = 0.90$	

$F_y = 50$ ksi

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes



W10

Shape		W10×											
Wt/ft		112		100		88		77		68		60	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	986	1480	880	1320	775	1170	678	1020	598	899	528	794
	6	936	1410	834	1250	734	1100	641	963	565	850	499	750
	7	918	1380	818	1230	720	1080	628	944	554	833	489	734
	8	898	1350	800	1200	703	1060	614	922	541	813	477	717
	9	876	1320	780	1170	685	1030	598	898	527	792	464	698
	10	852	1280	758	1140	666	1000	580	872	511	768	450	677
	11	826	1240	734	1100	645	969	561	844	495	744	436	655
	12	799	1200	709	1070	623	936	542	814	477	717	420	631
	13	770	1160	683	1030	600	901	521	783	459	690	404	606
	14	740	1110	656	986	575	865	500	751	440	661	387	581
	15	709	1070	628	944	551	827	477	718	420	632	369	555
	16	678	1020	600	901	525	789	455	684	400	602	351	528
	17	646	970	571	858	499	751	432	650	380	571	333	501
	18	613	922	542	814	474	712	409	615	360	541	315	474
	19	581	873	512	770	448	673	386	581	339	510	297	447
	20	549	825	483	726	422	634	364	546	319	480	279	420
	22	485	729	426	640	371	558	319	479	280	421	244	367
	24	424	637	371	558	323	485	276	415	242	364	211	317
	26	365	549	319	479	277	416	236	355	207	311	180	270
	28	315	473	275	413	239	358	204	306	178	268	155	233
	30	274	412	239	360	208	312	178	267	155	234	135	203
	32	241	362	210	316	183	274	156	234	137	205	119	179
	34	214	321	186	280	162	243	138	208	121	182	105	158
	36	191	286	166	250	144	217	123	185	108	162	93.9	141
	38	171	257	149	224	129	195	111	166	96.9	146	84.2	127
	40	154	232	135	202	117	176	99.8	150	87.4	131	76.0	114
Properties													
P_{wo} (kips)		220	330	184	275	150	225	121	182	99.5	149	82.6	124
P_{wi} (kips/in.)		25.2	37.8	22.7	34.0	20.2	30.3	17.7	26.5	15.7	23.5	14.0	21.0
P_{wb} (kips)		948	1420	692	1040	488	733	328	493	229	344	163	245
P_{fb} (kips)		292	439	235	353	183	276	142	213	111	167	86.5	130
L_p (ft)		9.47		9.36		9.29		9.18		9.15		9.08	
L_r (ft)		64.3		57.7		51.1		45.2		40.6		36.6	
A_g (in.^{2})		32.9		29.4		25.9		22.6		20.0		17.6	
I_x (in.^{4})		716		623		534		455		394		341	
I_y (in.^{4})		236		207		179		154		134		116	
r_y (in.)		2.68		2.65		2.63		2.60		2.59		2.57	
Ratio r_x/r_y		1.74		1.74		1.73		1.73		1.71		1.71	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		20500		17800		15300		13000		11300		9760	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		6750		5920		5120		4410		3840		3320	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

W10

Table 4–1 (continued)
Available Strength in Axial Compression, kips
W Shapes

F_y = 50 ksi

Shape		W10×									
Wt/ft		54		49		45		39		33	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	474	712	432	649	397	597	343	516	291	437
	6	447	672	407	612	361	543	312	469	263	395
	7	438	658	399	599	349	525	301	452	253	381
	8	428	643	389	585	336	505	289	435	243	365
	9	416	625	378	569	321	483	276	415	232	348
	10	404	607	367	551	306	460	263	395	220	330
	11	390	586	355	533	290	435	248	373	207	311
	12	376	565	341	513	273	410	233	351	194	292
	13	361	543	328	493	256	384	218	328	181	272
	14	346	520	314	471	238	358	203	305	168	253
	15	330	496	299	450	221	332	188	282	155	233
	16	314	472	284	428	204	306	173	260	142	213
	17	298	448	270	405	187	281	158	238	130	195
	18	282	423	255	383	171	256	144	216	117	177
	19	265	399	240	360	155	233	130	195	106	159
	20	249	375	225	338	140	210	117	176	95.4	143
	22	218	327	196	295	116	174	97.0	146	78.8	118
	24	188	282	169	254	97.1	146	81.5	122	66.2	99.5
	26	160	241	144	216	82.7	124	69.4	104	56.4	84.8
	28	138	208	124	186	71.3	107	59.9	90.0	48.7	73.1
	30	120	181	108	162	62.1	93.4	52.2	78.4	42.4	63.7
	32	106	159	94.9	143	54.6	82.1	45.8	68.9	37.2	56.0
	34	93.7	141	84.0	126						
	36	83.6	126	75.0	113						
	38	75.0	113	67.3	101						
	40	67.7	102	60.7	91.3						

Properties

P_{wo} (kips)	68.8	103	60.1	90.1	65.3	98.0	54.1	81.1	45.2	67.8
P_{wi} (kips/in.)	12.3	18.5	11.3	17.0	11.7	17.5	10.5	15.8	9.67	14.5
P_{wb} (kips)	112	168	86.5	130	94.4	142	68.8	103	53.7	80.7
P_{fb} (kips)	70.8	106	58.7	88.2	71.9	108	52.6	79.0	35.4	53.2
L_p (ft)	9.04		8.97		7.10		6.99		6.85	
L_r (ft)	33.7		31.6		26.9		24.2		21.8	
A_g (in.^{2})	15.8		14.4		13.3		11.5		9.71	
I_x (in.^{4})	303		272		248		209		171	
I_y (in.^{4})	103		93.4		53.4		45.0		36.6	
r_y (in.)	2.56		2.54		2.01		1.98		1.94	
Ratio r_x/r_y	1.71		1.71		2.15		2.16		2.16	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	8670		7790		7100		5980		4890	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	2950		2670		1530		1290		1050	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 4–1 (continued)

Available Strength in Axial Compression, kips

W Shapes

W8

Shape	W8×											
Wt/ft	67		58		48		40		35		31	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y												
0	**589**	886	**512**	769	**422**	634	**351**	528	**308**	463	**273**	410
6	**542**	814	**469**	706	**387**	581	**321**	482	**281**	422	**249**	374
7	**525**	790	**455**	684	**375**	563	**310**	467	**272**	408	**241**	362
8	**507**	762	**439**	660	**361**	543	**299**	449	**261**	393	**232**	348
9	**487**	733	**422**	634	**347**	521	**286**	430	**250**	376	**222**	333
10	**466**	701	**403**	606	**331**	497	**273**	410	**238**	358	**211**	317
11	**444**	667	**383**	576	**314**	473	**259**	389	**226**	340	**200**	300
12	**421**	632	**363**	545	**297**	447	**244**	367	**213**	320	**188**	283
13	**397**	596	**342**	514	**280**	420	**229**	344	**200**	300	**177**	265
14	**372**	560	**320**	482	**262**	394	**214**	322	**187**	280	**165**	248
15	**348**	523	**299**	449	**244**	367	**199**	299	**173**	260	**153**	230
16	**323**	486	**278**	417	**226**	340	**184**	276	**160**	241	**141**	212
17	**299**	450	**257**	386	**209**	314	**169**	254	**147**	221	**130**	195
18	**276**	415	**236**	355	**192**	288	**155**	233	**135**	202	**118**	178
19	**253**	380	**216**	325	**175**	263	**141**	212	**122**	184	**108**	162
20	**231**	347	**197**	296	**159**	239	**127**	192	**111**	166	**97.1**	146
22	**191**	287	**162**	244	**132**	198	**105**	158	**91.4**	137	**80.3**	121
24	**160**	241	**137**	205	**111**	166	**88.5**	133	**76.8**	115	**67.4**	101
26	**137**	205	**116**	175	**94.2**	142	**75.4**	113	**65.4**	98.3	**57.5**	86.4
28	**118**	177	**100**	151	**81.2**	122	**65.0**	97.8	**56.4**	84.8	**49.6**	74.5
30	**103**	154	**87.4**	131	**70.7**	106	**56.7**	85.2	**49.1**	73.9	**43.2**	64.9
32	**90.2**	136	**76.8**	115	**62.2**	93.4	**49.8**	74.8	**43.2**	64.9	**37.9**	57.0
34	**79.9**	120	**68.0**	102	**55.1**	82.8	**44.1**	66.3				

Properties

	67		58		48		40		35		31	
P_{wo} (kips)	**126**	189	**102**	154	**71.9**	108	**57.2**	85.9	**45.9**	68.9	**39.4**	59.1
P_{wi} (kips/in.)	**19.0**	28.5	**17.0**	25.5	**13.3**	20.0	**12.0**	18.0	**10.3**	15.5	**9.50**	14.3
P_{wb} (kips)	**505**	760	**362**	544	**175**	262	**127**	191	**81.3**	122	**63.2**	94.9
P_{fb} (kips)	**164**	246	**123**	185	**87.8**	132	**58.7**	88.2	**45.9**	68.9	**35.4**	53.2
L_p (ft)	7.49		7.42		7.35		7.21		7.17		7.18	
L_r (ft)	47.7		41.7		35.2		29.9		27.0		24.8	
A_g (in.^{2})	19.7		17.1		14.1		11.7		10.3		9.12	
I_x (in.^{4})	272		228		184		146		127		110	
I_y (in.^{4})	88.6		75.1		60.9		49.1		42.6		37.1	
r_y (in.)	2.12		2.10		2.08		2.04		2.03		2.02	
Ratio r_x/r_y	1.75		1.74		1.74		1.73		1.73		1.72	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})	7790		6530		5270		4180		3630		3150	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})	2540		2150		1740		1410		1220		1060	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.

Table 4–2
Available Strength in Axial Compression, kips
HP Shapes

HP14–HP12

F_y = 50 ksi

Shape		HP14×								HP12×			
Wt/ft		117		102		89		73^c		84		74	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y	**0**	1030	1550	899	1350	782	1180	622	935	737	1110	653	981
	6	1000	1500	873	1310	759	1140	604	908	706	1060	625	939
	7	990	1490	863	1300	751	1130	597	898	695	1040	615	924
	8	978	1470	853	1280	741	1110	590	886	682	1020	603	907
	9	964	1450	841	1260	731	1100	581	874	668	1000	591	888
	10	950	1430	828	1240	719	1080	572	860	653	981	577	867
	11	933	1400	813	1220	706	1060	562	845	636	956	562	845
	12	916	1380	798	1200	693	1040	551	829	619	930	547	822
	13	898	1350	782	1170	678	1020	540	812	600	902	530	797
	14	878	1320	764	1150	663	996	528	793	581	873	513	770
	15	857	1290	746	1120	647	972	515	774	561	842	495	743
	16	836	1260	727	1090	630	947	502	754	540	811	476	715
	17	814	1220	707	1060	613	921	488	734	518	779	457	687
	18	791	1190	687	1030	595	894	474	713	497	747	438	658
	19	767	1150	666	1000	577	867	460	691	475	714	418	628
	20	743	1120	645	969	558	839	445	669	453	681	398	599
	22	694	1040	602	904	520	781	415	623	409	615	359	540
	24	644	967	557	838	481	723	384	577	366	549	321	482
	26	593	891	513	771	442	664	353	530	324	486	283	426
	28	543	816	469	705	403	606	322	484	283	426	248	372
	30	494	742	426	640	366	550	292	439	247	371	216	324
	32	446	671	384	577	329	495	263	396	217	326	190	285
	34	400	602	344	517	294	442	235	354	192	289	168	252
	36	357	537	307	461	262	394	210	316	171	258	150	225
	38	321	482	275	414	235	354	188	283	154	231	134	202
	40	289	435	248	373	212	319	170	256	139	209	121	182
Properties													
P_{wo} (kips)		201	302	162	242	135	202	99.9	150	157	235	132	199
P_{wi} (kips/in.)		26.8	40.3	23.5	35.3	20.5	30.8	16.8	25.3	22.8	34.3	20.2	30.3
P_{wb} (kips)		792	1190	532	799	353	531	195	294	573	861	394	593
P_{fb} (kips)		121	182	93.0	140	70.8	106	47.7	71.7	87.8	132	69.6	105
L_p (ft)		12.9		15.5		17.9		21.3		10.4		11.9	
L_r (ft)		50.5		45.7		41.7		37.6		41.4		37.9	
A_g (in.^{2})		34.4		30.0		26.1		21.4		24.6		21.8	
I_x (in.^{4})		1220		1050		904		729		650		569	
I_y (in.^{4})		443		380		326		261		213		186	
r_y (in.)		3.59		3.56		3.53		3.49		2.94		2.92	
Ratio r_x/r_y		1.66		1.66		1.67		1.67		1.75		1.75	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		34900		30100		25900		20900		18600		16300	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		12700		10900		9330		7470		6100		5320	

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

F_y = 50 ksi

Table 4–2 (continued)
Available Strength in Axial Compression, kips
HP Shapes

HP12–HP8

Shape		HP12×				HP10×				HP8×	
Wt/ft		63		53		57		42		36	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	551	829	460	692	502	755	370	556	316	476
	6	527	792	440	661	472	709	347	521	286	430
	7	518	779	432	650	461	693	338	509	276	415
	8	508	764	424	638	449	675	329	495	265	398
	9	497	748	415	624	436	655	319	480	253	380
	10	486	730	405	609	421	633	309	464	240	361
	11	473	711	394	593	406	611	297	446	226	340
	12	459	690	383	576	390	586	285	428	212	319
	13	445	669	371	558	373	561	272	409	198	298
	14	430	646	359	539	356	535	259	390	184	276
	15	414	623	346	519	338	509	246	370	170	255
	16	398	599	332	499	321	482	233	350	156	234
	17	382	574	318	479	303	455	219	329	142	214
	18	365	549	305	458	285	428	206	309	129	194
	19	349	524	291	437	267	401	192	289	116	175
	20	332	499	276	416	249	374	179	269	105	158
	22	298	448	248	373	215	323	154	231	86.7	130
	24	265	399	221	332	182	274	130	195	72.8	109
	26	234	351	195	292	155	234	111	167	62.0	93.3
	28	203	306	169	254	134	201	95.5	144	53.5	80.4
	30	177	266	147	221	117	176	83.2	125	46.6	70.0
	32	156	234	130	195	103	154	73.1	110	41.0	61.6
	34	138	207	115	172	90.9	137	64.8	97.4		
	36	123	185	102	154	81.1	122	57.8	86.9		
	38	110	166	91.8	138	72.8	109	51.9	78.0		
	40	99.6	150	82.9	125	65.7	98.7	46.8	70.4		
Properties											
P_{wo} (kips)		107	161	81.6	122	118	177	77.8	117	83.4	125
P_{wi} (kips/in.)		17.2	25.8	14.5	21.8	18.8	28.3	13.8	20.8	14.8	22.3
P_{wb} (kips)		243	366	147	220	397	597	157	237	241	362
P_{fb} (kips)		49.6	74.6	35.4	53.2	59.7	89.8	33.0	49.6	37.1	55.7
L_p (ft)		14.4		16.7		8.65		12.3		6.90	
L_r (ft)		34.0		31.1		34.9		28.3		27.3	
A_g (in.^{2})		18.4		15.5		16.8		12.4		10.6	
I_x (in.^{4})		472		393		294		210		119	
I_y (in.^{4})		153		127		101		71.7		40.3	
r_y (in.)		2.88		2.86		2.45		2.41		1.95	
Ratio r_x/r_y		1.76		1.76		1.71		1.71		1.72	
$P_{ex}(KL^2)/10^4$ (k-in.^{2})		13500		11200		8410		6010		3410	
$P_{ey}(KL^2)/10^4$ (k-in.^{2})		4380		3630		2890		2050		1150	

ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

Table 4–3
Available Strength in Axial Compression, kips
Rectangular HSS

HSS20–HSS16

$F_y = 46$ ksi

Shape		HSS20×12×								HSS16×12×			
		5/8		$1/2^c$		$3/8^c$		$5/16^c$		5/8		1/2	
t_{design}, in.		0.581		0.465		0.349		0.291		0.581		0.465	
Wt/ft		127		103		78.4		65.8		110		89.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	963	1450	741	1110	495	745	374	562	835	1260	678	1020
	6	949	1430	733	1100	491	738	371	558	823	1240	668	1000
	7	945	1420	731	1100	489	735	370	557	818	1230	665	999
	8	939	1410	727	1090	487	733	369	555	813	1220	661	993
	9	933	1400	724	1090	485	730	368	553	807	1210	656	986
	10	926	1390	720	1080	483	726	367	551	801	1200	651	978
	11	918	1380	715	1070	480	722	366	549	794	1190	645	970
	12	909	1370	710	1070	478	718	364	547	786	1180	639	961
	13	900	1350	705	1060	475	713	362	544	778	1170	633	951
	14	891	1340	699	1050	471	708	360	542	769	1160	626	940
	15	881	1320	693	1040	468	703	358	539	760	1140	618	929
	16	870	1310	686	1030	464	697	356	536	750	1130	610	918
	17	859	1290	679	1020	460	691	354	532	740	1110	602	905
	18	847	1270	672	1010	456	685	352	529	729	1100	594	892
	19	834	1250	664	998	451	679	349	525	718	1080	585	879
	20	821	1230	656	986	447	672	346	521	706	1060	575	865
	21	808	1210	648	973	442	664	344	516	694	1040	566	850
	22	794	1190	639	960	437	657	341	512	682	1020	556	836
	23	780	1170	630	946	432	649	337	507	669	1010	546	820
	24	766	1150	620	932	426	641	334	502	656	986	535	805
	25	751	1130	610	917	420	632	330	496	642	966	525	789
	26	736	1110	600	902	415	623	326	490	629	945	514	772
	27	721	1080	588	883	409	614	322	483	615	924	503	756
	28	705	1060	575	864	402	605	317	477	601	903	492	739
	29	689	1040	563	846	396	595	313	470	587	882	480	722
	30	673	1010	550	826	389	585	308	463	572	860	469	704
	32	641	963	524	787	376	564	298	448	543	817	445	669
	34	608	914	498	748	361	543	288	433	514	773	422	634
	36	575	864	471	708	346	520	277	417	485	729	398	599
	38	542	815	445	669	331	497	266	400	456	685	375	563
	40	510	766	419	629	315	473	254	383	427	641	352	528
Properties													
A_g (in.^{2})		35.0		28.3		21.5		18.1		30.3		24.6	
I_x (in.^{4})		1880		1550		1200		1010		1090		904	
I_y (in.^{4})		851		705		547		464		700		581	
r_x/r_y		1.48		1.48		1.48		1.48		1.25		1.25	
r_y (in.)		4.93		4.99		5.04		5.07		4.80		4.86	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 46$ ksi.

Table 4-3 (continued)

Available Strength in Axial Compression, kips

Rectangular HSS

F_y = 46 ksi

HSS16

Shape		HSS16×12×				HSS16×8×							
		$^3/_8{}^c$		$^5/_{16}{}^c$		$^5/_8$		$^1/_2$		$^3/_8{}^c$		$^5/_{16}{}^c$	
t_{design}, in.		0.349		0.291		0.581		0.465		0.349		0.291	
Wt/ft		68.3		57.4		93.1		75.9		58.1		48.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	480	722	365	549	707	1060	576	865	403	606	311	467
	6	475	714	362	544	684	1030	558	838	395	593	304	458
	7	473	711	361	543	676	1020	551	829	391	588	302	454
	8	471	708	360	541	667	1000	544	818	388	583	300	451
	9	468	704	359	539	657	987	536	806	384	577	297	446
	10	466	700	357	537	646	971	527	792	379	570	294	442
	11	463	696	355	534	634	952	518	778	374	562	290	436
	12	459	691	354	531	621	933	507	762	369	554	287	431
	13	456	685	352	528	607	912	496	746	363	545	283	425
	14	452	679	349	525	592	890	485	728	356	536	278	418
	15	448	673	347	522	577	867	472	710	350	525	273	411
	16	444	667	345	518	561	843	460	691	342	515	268	404
	17	439	660	342	514	544	818	447	671	335	503	263	396
	18	434	653	339	510	527	792	433	651	327	491	258	387
	19	429	645	336	505	510	766	419	630	319	479	252	379
	20	424	637	333	500	492	739	405	609	310	466	246	370
	21	418	628	330	495	474	712	391	587	301	452	240	360
	22	412	620	326	490	456	685	376	565	291	437	233	350
	23	406	611	322	483	438	658	362	544	280	421	226	340
	24	400	601	317	477	419	630	347	522	269	404	219	330
	25	393	591	313	470	401	603	332	500	258	388	212	319
	26	387	581	308	462	383	576	318	478	247	371	205	308
	27	380	571	303	455	365	549	303	456	236	355	197	296
	28	373	560	298	447	347	522	289	434	225	339	189	284
	29	366	549	292	439	330	496	275	413	215	323	181	273
	30	358	538	287	431	313	470	261	392	204	307	173	260
	32	342	514	276	415	279	420	234	352	184	276	156	235
	34	325	488	264	397	248	372	208	313	164	246	140	210
	36	307	461	252	379	221	332	186	279	146	220	125	187
	38	289	434	240	360	198	298	167	250	131	197	112	168
	40	272	408	227	341	179	269	150	226	118	178	101	152

Properties

A_g (in.^{2})	18.7	15.7	25.7	20.9	16.0	13.4
I_x (in.^{4})	702	595	815	679	531	451
I_y (in.^{4})	452	384	274	230	181	155
r_x/r_y	1.25	1.25	1.72	1.72	1.71	1.71
r_y (in.)	4.91	4.94	3.27	3.32	3.37	3.40

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS16–HSS14

F_y = 46 ksi

Shape		HSS16×8×		HSS14×10×									
		$^1/_4{}^c$		$^5/_8$		$^1/_2$		$^3/_8{}^c$		$^5/_{16}{}^c$		$^1/_4{}^c$	
t_{design}, in.		0.233		0.581		0.465		0.349		0.291		0.233	
Wt/ft		39.5		93.1		75.9		58.1		48.9		39.5	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	225	338	707	1060	576	865	431	647	336	505	238	358
	6	221	332	692	1040	563	847	423	637	331	498	235	354
	7	220	330	686	1030	559	840	421	633	329	495	235	353
	8	218	328	680	1020	554	833	418	628	327	492	234	351
	9	216	325	673	1010	549	825	415	623	325	488	232	349
	10	214	322	665	1000	542	815	411	618	322	484	231	347
	11	212	319	657	987	536	805	407	611	319	480	230	345
	12	210	315	648	973	528	794	402	605	316	475	228	343
	13	207	311	638	959	521	783	398	598	313	470	226	340
	14	204	307	627	943	512	770	392	590	309	465	224	337
	15	201	302	616	926	504	757	386	580	305	459	222	334
	16	198	298	605	909	494	743	379	569	301	453	220	331
	17	195	293	593	891	485	729	372	559	297	446	218	327
	18	191	287	580	872	475	714	364	547	292	439	215	324
	19	187	282	567	853	464	698	356	536	287	432	213	320
	20	183	276	554	833	454	682	349	524	282	424	210	316
	21	179	270	540	812	443	666	340	512	277	416	207	311
	22	175	263	526	791	432	649	332	499	271	408	204	307
	23	171	257	512	769	420	632	323	486	266	399	201	302
	24	166	250	497	748	409	614	315	473	260	391	197	296
	25	162	243	483	726	397	597	306	460	254	381	193	290
	26	157	236	468	703	385	579	297	446	248	372	189	283
	27	152	228	453	681	373	561	288	433	241	362	184	277
	28	147	220	438	659	361	543	279	419	235	353	180	270
	29	142	213	423	636	349	525	270	406	228	343	175	264
	30	136	205	408	614	337	507	261	392	221	332	171	257
	32	125	188	378	569	313	471	243	365	206	309	161	242
	34	114	171	349	525	290	435	225	338	191	287	151	228
	36	102	154	321	482	266	400	207	312	176	265	141	213
	38	91.7	138	293	440	244	367	190	286	162	243	131	197
	40	82.8	124	266	400	222	334	174	261	148	222	121	181

Properties

	HSS16×8×$^1/_4$	HSS14×10×$^5/_8$	HSS14×10×$^1/_2$	HSS14×10×$^3/_8$	HSS14×10×$^5/_{16}$	HSS14×10×$^1/_4$
A_g (in.^{2})	10.8	25.7	20.9	16.0	13.4	10.8
I_x (in.^{4})	368	687	573	447	380	310
I_y (in.^{4})	127	407	341	267	227	186
r_x/r_y	1.70	1.30	1.30	1.29	1.29	1.29
r_y (in.)	3.42	3.98	4.04	4.09	4.12	4.14

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

F_y = 46 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS12

Shape		HSS12×10×								HSS12×8×			
		1/2		3/8		5/16^c		1/4^c		5/8		1/2	
t_{design}, in.		0.465		0.349		0.291		0.233		0.581		0.465	
Wt/ft		69.1		52.9		44.6		36.0		76.1		62.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	524	788	401	603	328	493	234	351	579	870	473	711
	6	513	771	392	590	322	484	231	347	559	840	457	688
	7	509	765	389	585	320	481	230	346	552	830	452	679
	8	504	758	386	580	318	478	229	344	544	818	446	670
	9	499	750	382	574	315	474	227	342	535	805	439	659
	10	493	741	378	567	312	469	226	340	526	790	431	648
	11	487	731	373	560	309	465	224	337	515	774	422	635
	12	480	721	368	553	306	459	223	335	504	757	413	621
	13	472	710	362	544	302	454	221	332	492	739	404	607
	14	465	698	356	536	298	448	219	328	479	720	394	592
	15	456	686	350	526	294	441	216	325	466	700	383	576
	16	448	673	344	517	289	434	214	321	452	679	372	559
	17	439	659	337	506	284	427	211	318	438	658	361	542
	18	429	645	330	496	278	418	209	313	423	636	349	525
	19	420	631	323	485	272	409	206	309	408	613	337	507
	20	410	616	315	474	266	400	203	304	393	591	325	489
	21	399	600	307	462	260	390	199	300	378	568	313	470
	22	389	584	300	450	253	380	196	294	362	545	301	452
	23	378	568	292	438	246	370	192	288	347	521	288	433
	24	367	552	283	426	240	360	187	281	331	498	276	414
	25	356	536	275	414	233	350	183	275	316	475	263	396
	26	345	519	267	401	226	339	178	268	301	452	251	377
	27	334	502	259	389	219	329	174	261	286	429	239	359
	28	323	485	250	376	212	318	169	254	271	407	227	341
	29	312	469	242	363	205	308	164	247	256	385	215	323
	30	301	452	233	351	198	297	159	239	242	364	204	306
	32	278	418	217	325	184	276	149	224	214	322	181	272
	34	257	386	200	301	170	255	138	208	190	285	160	241
	36	235	354	184	276	156	235	128	192	169	254	143	215
	38	215	323	168	253	143	215	117	176	152	228	128	193
	40	195	292	153	230	130	196	107	160	137	206	116	174

Properties

	1/2	3/8	5/16^c	1/4^c	5/8	1/2
A_g (in.^{2})	19.0	14.6	12.2	9.90	21.0	17.2
I_x (in.^{4})	395	310	264	216	397	333
I_y (in.^{4})	298	234	200	164	210	178
r_x/r_y	1.15	1.15	1.15	1.15	1.37	1.37
r_y (in.)	3.96	4.01	4.04	4.07	3.16	3.21

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

HSS12

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

F_y = 46 ksi

Shape		HSS12×8×								HSS12×6×			
		3/8		5/16^c		1/4^c		3/16^c		5/8		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.581		0.465	
Wt/ft		47.8		40.4		32.6		24.8		67.6		55.5	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	362	545	296	444	218	327	136	204	515	774	422	634
	6	351	527	288	433	213	320	134	201	484	728	398	598
	7	347	521	286	429	211	317	133	200	474	712	390	586
	8	342	514	283	425	209	314	132	199	462	694	380	571
	9	337	506	279	420	207	311	131	198	449	674	370	556
	10	331	498	275	414	204	307	130	196	435	653	359	539
	11	325	488	271	408	202	303	129	194	419	630	346	521
	12	318	478	267	401	199	299	128	192	403	606	334	502
	13	311	467	262	393	195	294	127	190	386	581	320	482
	14	303	456	256	385	192	289	125	188	369	555	307	461
	15	296	444	250	375	188	283	124	186	351	528	292	440
	16	287	432	243	365	184	277	122	183	333	501	278	418
	17	279	419	236	354	180	271	120	180	315	474	264	396
	18	270	406	229	344	176	265	118	177	297	446	249	374
	19	261	393	221	332	172	258	116	174	279	419	234	352
	20	252	379	214	321	167	251	114	171	261	392	220	331
	21	243	365	206	309	162	244	111	167	243	366	206	309
	22	234	351	198	298	157	236	109	164	226	340	192	288
	23	224	337	190	286	152	228	106	160	210	315	178	268
	24	215	323	183	274	147	220	103	155	193	290	165	248
	25	206	309	175	263	141	212	100	151	178	268	152	229
	26	196	295	167	251	136	204	96.9	146	165	247	141	211
	27	187	281	159	239	130	195	93.6	141	153	229	130	196
	28	178	267	152	228	124	186	90.1	135	142	213	121	182
	29	169	254	144	217	118	177	86.6	130	132	199	113	170
	30	160	241	137	205	112	168	83.1	125	124	186	106	159
	32	143	215	122	184	100	151	75.8	114	109	163	92.9	140
	34	127	191	109	163	89.2	134	68.4	103	96.2	145	82.3	124
	36	113	170	96.8	146	79.6	120	61.0	91.7	85.8	129	73.4	110
	38	102	153	86.9	131	71.4	107	54.7	82.3	77.0	116	65.9	99.0
	40	91.6	138	78.4	118	64.4	96.9	49.4	74.2	59.4	89.3	59.4	89.3

Properties

	3/8	5/16^c	1/4^c	3/16^c	5/8	1/2
A_g (in.^{2})	13.2	11.1	8.96	6.76	18.7	15.3
I_x (in.^{4})	262	224	184	140	321	271
I_y (in.^{4})	140	120	98.8	75.7	107	91.1
r_x/r_y	1.37	1.37	1.36	1.36	1.73	1.73
r_y (in.)	3.27	3.29	3.32	3.35	2.39	2.44

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

F_y = 46 ksi

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS12–HSS10

Shape		HSS12×6×								HSS10×8×			
		$^3/_8$		$^5/_{16}{}^c$		$^1/_4{}^c$		$^3/_{16}{}^c$		$^5/_8$		$^1/_2$	
t_{design}, in.		0.349		0.291		0.233		0.174		0.581		0.465	
Wt/ft		42.7		36.1		29.2		22.2		67.6		55.5	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	324	487	264	396	192	289	126	189	515	774	422	634
	6	306	460	253	380	185	278	122	183	496	746	407	612
	7	300	451	249	374	183	274	121	181	490	736	402	604
	8	293	441	244	367	180	270	119	179	483	725	396	596
	9	286	429	239	360	177	265	117	176	474	713	390	586
	10	277	417	234	351	173	260	115	173	465	699	382	575
	11	268	403	227	341	169	254	113	170	455	684	375	563
	12	259	389	219	329	165	248	111	166	445	669	366	551
	13	249	374	211	317	160	241	108	163	434	652	357	537
	14	239	359	202	304	156	234	105	159	422	634	348	523
	15	228	343	194	291	150	226	103	154	410	616	338	508
	16	217	326	185	278	145	218	99.6	150	397	597	328	493
	17	206	310	176	264	139	209	96.4	145	384	577	318	477
	18	195	294	166	250	133	201	93.0	140	370	557	307	461
	19	184	277	157	236	127	191	89.5	135	357	536	296	445
	20	173	261	148	223	121	182	85.9	129	343	515	285	428
	21	163	245	139	209	114	172	82.1	123	329	494	274	411
	22	152	229	130	196	107	161	78.2	118	315	473	262	394
	23	142	213	122	183	100	150	74.2	112	301	452	251	377
	24	132	198	113	170	93.3	140	70.1	105	287	431	240	360
	25	122	183	105	158	86.7	130	66.0	99.1	273	410	228	343
	26	113	169	96.9	146	80.1	120	61.7	92.7	259	389	217	326
	27	104	157	89.9	135	74.3	112	57.3	86.1	245	369	206	310
	28	97.1	146	83.6	126	69.1	104	53.3	80.0	232	349	195	294
	29	90.5	136	77.9	117	64.4	96.8	49.6	74.6	219	329	185	278
	30	84.6	127	72.8	109	60.2	90.4	46.4	69.7	206	310	174	262
	32	74.3	112	64.0	96.2	52.9	79.5	40.8	61.3	182	273	154	231
	34	65.9	99.0	56.7	85.2	46.8	70.4	36.1	54.3	161	242	136	205
	36	58.7	88.3	50.6	76.0	41.8	62.8	32.2	48.4	143	216	122	183
	38	52.7	79.2	45.4	68.2	37.5	56.4	28.9	43.5	129	194	109	164
	40	47.6	71.5	41.0	61.6	33.8	50.9	26.1	39.2	116	175	98.6	148

Properties

A_g (in.^{2})	11.8	9.92	8.03	6.06	18.7	15.3
I_x (in.^{4})	215	184	151	116	253	214
I_y (in.^{4})	72.9	62.8	51.9	40.0	178	151
r_x/r_y	1.72	1.71	1.71	1.70	1.19	1.19
r_y (in.)	2.49	2.52	2.54	2.57	3.09	3.14

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

HSS10

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

F_y = 46 ksi

Shape		HSS10×8×								HSS10×6×	
		3/8		5/16		1/4^c		3/16^c		5/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.581	
Wt/ft		42.7		36.1		29.2		22.2		59.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	324	487	273	411	212	318	133	201	451	678
	6	313	471	264	397	206	310	131	197	423	636
	7	309	465	261	392	204	307	130	196	413	621
	8	305	458	257	387	202	303	129	195	403	605
	9	300	451	253	381	199	300	128	193	391	587
	10	295	443	249	374	197	295	127	191	378	568
	11	289	434	244	367	193	291	126	189	364	547
	12	283	425	239	359	190	286	124	187	349	525
	13	276	415	233	351	187	280	123	185	334	502
	14	269	404	228	342	183	275	121	182	319	479
	15	262	393	221	333	179	269	119	179	303	455
	16	254	382	215	323	174	262	117	176	286	431
	17	246	370	209	314	170	255	115	173	270	406
	18	238	358	202	303	164	247	113	170	254	382
	19	230	346	195	293	159	239	111	166	238	357
	20	222	333	188	283	153	230	108	163	222	334
	21	213	320	181	272	148	222	105	159	206	310
	22	205	308	174	261	142	213	103	154	191	287
	23	196	295	167	251	136	205	99.4	149	177	265
	24	188	282	160	240	130	196	95.9	144	162	244
	25	179	269	152	229	125	187	92.4	139	149	225
	26	171	256	145	218	119	179	88.8	133	138	208
	27	162	244	138	208	113	170	85.2	128	128	192
	28	154	231	131	198	108	162	81.5	122	119	179
	29	146	219	125	187	102	154	77.7	117	111	167
	30	138	207	118	177	96.8	146	73.9	111	104	156
	32	122	184	105	158	86.4	130	66.3	99.6	91.2	137
	34	108	163	92.9	140	76.5	115	58.7	88.3	80.8	121
	36	96.7	145	82.9	125	68.2	103	52.4	78.8	72.0	108
	38	86.8	130	74.4	112	61.2	92.0	47.0	70.7	64.7	97.2
	40	78.3	118	67.1	101	55.3	83.1	42.4	63.8		

Properties

	3/8	5/16	1/4^c	3/16^c	5/8
A_g (in.^{2})	11.8	9.92	8.03	6.06	16.4
I_x (in.^{4})	169	145	119	91.4	201
I_y (in.^{4})	120	103	84.7	65.1	89.4
r_x/r_y	1.19	1.19	1.19	1.19	1.50
r_y (in.)	3.19	3.22	3.25	3.28	2.34

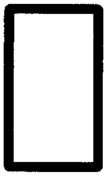
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^c Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 46 ksi

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS



HSS10

Shape		HSS10×6×									
		1/2		3/8		5/16		1/4^c		3/16^c	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174	
Wt/ft		48.7		37.6		31.8		25.8		19.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	371	557	286	429	241	362	186	279	123	185
	6	349	524	269	405	228	342	178	268	119	179
	7	341	513	264	396	223	335	175	263	117	176
	8	333	500	257	387	218	327	172	259	116	174
	9	323	486	250	376	212	319	168	253	114	171
	10	313	470	243	365	206	309	164	247	111	167
	11	302	454	235	353	199	299	160	241	109	164
	12	290	436	226	340	192	288	155	234	106	160
	13	278	418	217	326	184	277	150	226	103	155
	14	266	400	208	312	177	265	144	217	100	151
	15	253	380	198	298	169	253	138	207	97.0	146
	16	240	361	188	283	160	241	131	197	93.5	141
	17	227	341	179	268	152	229	125	187	90.0	135
	18	214	322	169	253	144	216	118	177	86.2	130
	19	201	302	159	239	136	204	111	167	82.4	124
	20	188	283	149	224	128	192	105	158	78.4	118
	21	175	264	139	210	120	180	98.4	148	74.3	112
	22	163	245	130	195	112	168	92.0	138	70.1	105
	23	151	227	121	182	104	156	85.8	129	65.8	98.8
	24	139	209	112	168	96.4	145	79.7	120	61.4	92.3
	25	128	193	103	155	89.0	134	73.7	111	56.9	85.6
	26	119	178	95.4	143	82.3	124	68.1	102	52.6	79.1
	27	110	165	88.5	133	76.3	115	63.2	95.0	48.8	73.4
	28	102	154	82.3	124	71.0	107	58.8	88.3	45.4	68.2
	29	95.3	143	76.7	115	66.1	99.4	54.8	82.3	42.3	63.6
	30	89.1	134	71.6	108	61.8	92.9	51.2	76.9	39.5	59.4
	32	78.3	118	63.0	94.6	54.3	81.6	45.0	67.6	34.7	52.2
	34	69.4	104	55.8	83.8	48.1	72.3	39.8	59.9	30.8	46.3
	36	61.9	93.0	49.8	74.8	42.9	64.5	35.5	53.4	27.5	41.3
	38	55.5	83.5	44.7	67.1	38.5	57.9	31.9	47.9	24.6	37.0
	40			40.3	60.6	34.8	52.3	28.8	43.3	22.2	33.4

Properties

	1/2	3/8	5/16	1/4^c	3/16^c
A_g (in.^{2})	13.5	10.4	8.76	7.10	5.37
I_x (in.^{4})	171	137	118	96.9	74.6
I_y (in.^{4})	76.8	61.8	53.3	44.1	34.1
r_x/r_y	1.49	1.49	1.48	1.48	1.48
r_y (in.)	2.39	2.44	2.47	2.49	2.52

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

Table 4–3 (continued)
Available Strength in Axial Compression, kips

F_y = 46 ksi

Rectangular HSS

HSS10–HSS9

Shape		HSS10×5×								HSS9×7×	
		3/8		5/16		1/4^c		3/16^c		5/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.581	
Wt/ft		35.1		29.7		24.1		18.4		59.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	266	400	225	338	173	260	114	171	451	678
	6	245	368	208	312	163	245	108	163	430	646
	7	238	358	202	303	159	240	106	160	422	634
	8	230	345	195	293	155	234	104	156	414	622
	9	221	332	188	282	151	227	102	153	404	608
	10	211	318	180	270	146	219	98.8	148	394	592
	11	201	303	171	258	140	211	95.7	144	383	576
	12	191	287	163	245	133	200	92.4	139	371	558
	13	180	271	154	231	126	190	88.9	134	359	539
	14	169	255	145	218	119	179	85.1	128	346	520
	15	159	238	136	204	112	168	81.1	122	333	500
	16	148	222	127	190	104	157	77.0	116	319	480
	17	137	206	118	177	96.9	146	72.6	109	305	459
	18	126	190	109	163	89.7	135	68.1	102	291	438
	19	116	174	99.9	150	82.7	124	63.5	95.4	277	416
	20	106	159	91.5	138	75.9	114	58.7	88.2	263	395
	21	96.1	144	83.3	125	69.3	104	53.8	80.8	249	374
	22	87.6	132	75.9	114	63.1	94.9	49.0	73.6	235	353
	23	80.1	120	69.4	104	57.8	86.8	44.8	67.4	221	332
	24	73.6	111	63.8	95.8	53.0	79.7	41.2	61.9	207	311
	25	67.8	102	58.8	88.3	48.9	73.5	37.9	57.0	194	291
	26	62.7	94.2	54.3	81.7	45.2	67.9	35.1	52.7	181	272
	27	58.1	87.4	50.4	75.7	41.9	63.0	32.5	48.9	168	253
	28	54.1	81.3	46.9	70.4	39.0	58.6	30.2	45.5	156	235
	29	50.4	75.7	43.7	65.6	36.3	54.6	28.2	42.4	146	219
	30	47.1	70.8	40.8	61.3	34.0	51.0	26.3	39.6	136	205
	32	41.4	62.2	35.9	53.9	29.8	44.8	23.2	34.8	120	180
	34	36.7	55.1	31.8	47.8	26.4	39.7	20.5	30.8	106	159
	36									94.6	142
	38									84.9	128
	40									76.6	115

Properties

A_g (in.^{2})	9.67	8.17	6.63	5.02	16.4
I_x (in.^{4})	120	104	85.8	66.2	174
I_y (in.^{4})	40.6	35.2	29.3	22.7	117
r_x/r_y	1.72	1.72	1.71	1.71	1.22
r_y (in.)	2.05	2.07	2.10	2.13	2.68

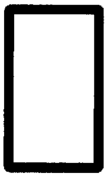
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS



HSS9

Shape		HSS9×7×									
		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4{}^c$		$^3/_{16}{}^c$	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174	
Wt/ft		48.7		37.6		31.8		25.8		19.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	371	557	286	429	241	362	195	292	129	194
	6	354	532	273	410	231	347	187	281	126	189
	7	348	523	269	404	227	341	184	277	125	187
	8	341	513	264	396	223	335	181	272	123	185
	9	334	502	258	388	218	328	177	267	122	183
	10	326	489	252	379	213	321	173	261	120	181
	11	317	476	246	369	208	313	169	254	118	178
	12	307	462	239	359	202	304	164	247	116	174
	13	298	447	231	348	196	295	160	240	113	170
	14	287	432	224	336	190	285	154	232	110	166
	15	277	416	216	324	183	275	149	224	107	161
	16	266	400	207	312	176	265	144	216	104	157
	17	255	383	199	299	169	254	138	208	101	152
	18	243	366	191	286	162	244	132	199	97.7	147
	19	232	349	182	273	155	233	127	190	94.2	142
	20	221	331	173	260	148	222	121	182	90.5	136
	21	209	314	165	247	140	211	115	173	86.8	130
	22	198	297	156	235	133	200	109	164	83.0	125
	23	187	280	147	222	126	190	104	156	79.1	119
	24	175	264	139	209	119	179	97.8	147	75.0	113
	25	165	247	131	197	112	169	92.2	139	70.8	106
	26	154	232	123	185	105	158	86.8	130	66.6	100
	27	144	216	115	173	98.7	148	81.4	122	62.6	94.1
	28	134	201	107	161	92.2	139	76.2	114	58.7	88.2
	29	125	187	99.8	150	85.9	129	71.0	107	54.7	82.2
	30	116	175	93.3	140	80.3	121	66.3	99.7	51.1	76.9
	32	102	154	82.0	123	70.6	106	58.3	87.6	44.9	67.5
	34	90.7	136	72.6	109	62.5	93.9	51.6	77.6	39.8	59.8
	36	80.9	122	64.8	97.3	55.7	83.8	46.1	69.2	35.5	53.4
	38	72.6	109	58.1	87.4	50.0	75.2	41.3	62.1	31.9	47.9
	40	65.5	98.5	52.5	78.8	45.2	67.9	37.3	56.1	28.8	43.2

Properties

A_g (in.^{2})	13.5	10.4	8.76	7.10	5.37
I_x (in.^{4})	149	119	102	84.1	64.7
I_y (in.^{4})	100	80.4	69.2	57.2	44.1
r_x/r_y	1.22	1.21	1.21	1.21	1.21
r_y (in.)	2.73	2.78	2.81	2.84	2.87

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS9

F_y = 46 ksi

Shape		HSS9×5×											
		5/8		1/2		3/8		5/16		1/4^c		3/16^c	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Wt/ft		50.6		41.9		32.5		27.6		22.4		17.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	387	582	319	480	247	371	209	314	169	254	112	169
	6	352	529	292	439	227	341	192	289	157	236	106	160
	7	340	512	283	425	220	331	187	281	152	229	104	156
	8	327	492	273	410	212	319	180	271	147	221	102	153
	9	313	470	261	393	204	307	174	261	142	213	98.9	149
	10	298	448	249	375	195	293	166	250	136	204	95.9	144
	11	282	424	237	355	186	279	158	238	129	195	92.7	139
	12	265	399	223	336	176	264	150	226	123	185	89.2	134
	13	249	374	210	316	166	249	142	213	116	175	85.4	128
	14	232	348	196	295	156	234	133	200	109	164	81.4	122
	15	215	323	183	275	145	218	125	187	103	154	77.2	116
	16	198	298	169	254	135	203	116	174	95.6	144	72.8	109
	17	182	273	156	234	125	188	108	162	88.8	133	68.3	103
	18	166	249	143	215	115	173	99.2	149	82.1	123	63.4	95.2
	19	150	226	130	196	105	158	91.1	137	75.6	114	58.4	87.8
	20	136	204	118	177	96.1	144	83.3	125	69.2	104	53.7	80.6
	21	123	185	107	161	87.2	131	75.6	114	63.0	94.7	49.0	73.6
	22	112	168	97.5	147	79.4	119	68.9	104	57.4	86.3	44.6	67.1
	23	103	154	89.2	134	72.7	109	63.1	94.8	52.5	79.0	40.8	61.4
	24	94.2	142	81.9	123	66.7	100	57.9	87.0	48.2	72.5	37.5	56.4
	25	86.8	130	75.5	113	61.5	92.4	53.4	80.2	44.5	66.8	34.6	51.9
	26	80.2	121	69.8	105	56.9	85.5	49.3	74.2	41.1	61.8	31.9	48.0
	27	74.4	112	64.7	97.3	52.7	79.2	45.8	68.8	38.1	57.3	29.6	44.5
	28	69.2	104	60.2	90.5	49.0	73.7	42.6	64.0	35.4	53.3	27.5	41.4
	29	64.5	96.9	56.1	84.3	45.7	68.7	39.7	59.6	33.0	49.7	25.7	38.6
	30	60.3	90.6	52.4	78.8	42.7	64.2	37.1	55.7	30.9	46.4	24.0	36.1
	32	53.0	79.6	46.1	69.3	37.5	56.4	32.6	49.0	27.1	40.8	21.1	31.7
	34							28.9	43.4	24.0	36.1	18.7	28.1

Properties

	5/8	1/2	3/8	5/16	1/4^c	3/16^c
A_g (in.^{2})	14.0	11.6	8.97	7.59	6.17	4.67
I_x (in.^{4})	133	115	92.5	79.8	66.1	51.1
I_y (in.^{4})	52.0	45.2	36.8	32.0	26.6	20.7
r_x/r_y	1.60	1.59	1.58	1.58	1.58	1.57
r_y (in.)	1.92	1.97	2.03	2.05	2.08	2.10

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 46 ksi

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS8

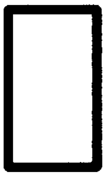
Shape		HSS8×6×									
		5/8		1/2		3/8		5/16		1/4	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233	
Wt/ft		50.6		41.9		32.5		27.6		22.4	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	387	582	319	480	247	371	209	314	170	255
	6	362	544	299	450	232	349	197	296	160	241
	7	353	530	293	440	227	341	193	289	157	236
	8	343	516	285	428	221	333	188	282	153	230
	9	332	499	276	415	215	323	183	274	149	224
	10	321	482	267	401	208	313	177	266	144	217
	11	308	463	257	386	201	302	171	257	139	209
	12	295	444	247	371	193	290	164	247	134	202
	13	282	423	236	354	185	278	157	237	129	193
	14	268	402	225	338	177	265	150	226	123	185
	15	253	381	213	321	168	252	143	215	117	176
	16	239	359	202	303	159	239	136	204	112	168
	17	225	338	190	286	150	226	129	193	106	159
	18	210	316	179	268	142	213	121	182	99.8	150
	19	196	295	167	251	133	200	114	171	93.9	141
	20	182	274	156	234	124	187	107	161	88.1	132
	21	169	254	145	217	116	174	99.7	150	82.3	124
	22	156	234	134	201	108	162	92.8	139	76.7	115
	23	143	214	123	185	99.7	150	86.0	129	71.3	107
	24	131	197	113	170	91.7	138	79.4	119	65.9	99.1
	25	121	182	104	157	84.5	127	73.1	110	60.8	91.3
	26	112	168	96.6	145	78.2	117	67.6	102	56.2	84.4
	27	104	156	89.5	135	72.5	109	62.7	94.3	52.1	78.3
	28	96.3	145	83.3	125	67.4	101	58.3	87.6	48.4	72.8
	29	89.8	135	77.6	117	62.8	94.4	54.4	81.7	45.1	67.9
	30	83.9	126	72.5	109	58.7	88.2	50.8	76.3	42.2	63.4
	32	73.7	111	63.7	95.8	51.6	77.6	44.6	67.1	37.1	55.7
	34	65.3	98.1	56.5	84.9	45.7	68.7	39.5	59.4	32.8	49.4
	36	58.2	87.5	50.4	75.7	40.8	61.3	35.3	53.0	29.3	44.0
	38			45.2	67.9	36.6	55.0	31.7	47.6	26.3	39.5
	40							28.6	42.9	23.7	35.7

Properties

	5/8	1/2	3/8	5/16	1/4
A_g (in.^{2})	14.0	11.6	8.97	7.59	6.17
I_x (in.^{4})	114	98.2	79.1	68.3	56.6
I_y (in.^{4})	72.3	62.5	50.6	43.8	36.4
r_x/r_y	1.26	1.25	1.25	1.25	1.25
r_y (in.)	2.27	2.32	2.38	2.40	2.43

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.



HSS8

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

$F_y = 46$ ksi

Shape		HSS8×6×		HSS8×4×							
		$^3/_{16}$^{c}		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$	
t_{design}, in.		0.174		0.581		0.465		0.349		0.291	
Wt/ft		17.1		42.1		35.1		27.4		23.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	**0**	**120**	180	**323**	485	**268**	403	**209**	314	**177**	266
	6	**114**	172	**277**	416	**232**	349	**182**	274	**155**	234
	7	**113**	169	**262**	394	**221**	332	**174**	261	**148**	223
	8	**110**	166	**246**	370	**208**	312	**164**	247	**140**	211
	9	**108**	163	**229**	344	**194**	292	**154**	232	**132**	198
	10	**106**	159	**211**	317	**180**	270	**143**	216	**123**	185
	11	**103**	154	**193**	290	**165**	249	**133**	199	**114**	172
	12	**99.6**	150	**175**	263	**151**	227	**122**	183	**105**	158
	13	**96.4**	145	**157**	236	**137**	205	**111**	167	**95.9**	144
	14	**92.9**	140	**140**	211	**123**	184	**100**	151	**87.0**	131
	15	**89.2**	134	**124**	186	**109**	164	**89.9**	135	**78.3**	118
	16	**85.3**	128	**109**	163	**96.3**	145	**79.9**	120	**70.0**	105
	17	**80.9**	122	**96.2**	145	**85.3**	128	**70.8**	106	**62.0**	93.2
	18	**76.5**	115	**85.8**	129	**76.1**	114	**63.1**	94.9	**55.3**	83.1
	19	**72.1**	108	**77.0**	116	**68.3**	103	**56.7**	85.2	**49.6**	74.6
	20	**67.7**	102	**69.5**	105	**61.7**	92.7	**51.1**	76.9	**44.8**	67.3
	21	**63.4**	95.3	**63.1**	94.8	**55.9**	84.1	**46.4**	69.7	**40.6**	61.1
	22	**59.2**	88.9	**57.5**	86.4	**51.0**	76.6	**42.3**	63.5	**37.0**	55.6
	23	**55.0**	82.7	**52.6**	79.0	**46.6**	70.1	**38.7**	58.1	**33.9**	50.9
	24	**51.0**	76.7	**48.3**	72.6	**42.8**	64.4	**35.5**	53.4	**31.1**	46.7
	25	**47.1**	70.7	**44.5**	66.9	**39.5**	59.3	**32.7**	49.2	**28.7**	43.1
	26	**43.5**	65.4			**36.5**	54.8	**30.3**	45.5	**26.5**	39.8
	27	**40.4**	60.6							**24.6**	36.9
	28	**37.5**	56.4								
	29	**35.0**	52.6								
	30	**32.7**	49.1								
	32	**28.7**	43.2								
	34	**25.4**	38.2								
	36	**22.7**	34.1								
	38	**20.4**	30.6								
	40	**18.4**	27.6								

Properties

	HSS8×6×$^3/_{16}$	HSS8×4×$^5/_8$	HSS8×4×$^1/_2$	HSS8×4×$^3/_8$	HSS8×4×$^5/_{16}$
A_g (in.^{2})	4.67	11.7	9.74	7.58	6.43
I_x (in.^{4})	43.7	82.0	71.8	58.7	51.0
I_y (in.^{4})	28.2	26.6	23.6	19.6	17.2
r_x/r_y	1.25	1.75	1.74	1.73	1.72
r_y (in.)	2.46	1.51	1.56	1.61	1.63

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with $F_y = 46$ ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 46 ksi

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS8–HSS7

Shape		HSS8×4×						HSS7×5×			
		$^{1}/_{4}$		$^{3}/_{16}{}^{c}$		$^{1}/_{8}{}^{c}$		$^{1}/_{2}$		$^{3}/_{8}$	
t_{design}, in.		0.233		0.174		0.116		0.465		0.349	
Wt/ft		19.0		14.5		9.85		35.1		27.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	144	217	100	151	55.9	84.0	268	403	209	314
	6	127	191	91.8	138	52.1	78.3	244	367	191	287
	7	121	182	88.7	133	50.8	76.3	236	354	185	277
	8	115	173	85.3	128	49.2	74.0	226	340	178	267
	9	108	163	81.4	122	47.5	71.4	216	325	170	256
	10	101	152	77.2	116	45.6	68.5	206	309	162	244
	11	94.2	142	72.5	109	43.5	65.4	195	293	154	232
	12	86.9	131	67.1	101	41.3	62.1	183	275	145	219
	13	79.6	120	61.6	92.6	39.0	58.6	171	258	137	205
	14	72.4	109	56.2	84.4	36.5	54.8	160	240	128	192
	15	65.4	98.3	50.9	76.5	33.9	50.9	148	222	119	178
	16	58.6	88.1	45.8	68.8	31.2	46.9	136	205	110	165
	17	52.1	78.3	40.8	61.4	28.4	42.6	125	188	101	152
	18	46.5	69.8	36.4	54.7	25.5	38.3	114	171	92.6	139
	19	41.7	62.7	32.7	49.1	22.8	34.3	103	155	84.4	127
	20	37.6	56.6	29.5	44.3	20.6	31.0	92.9	140	76.4	115
	21	34.1	51.3	26.8	40.2	18.7	28.1	84.3	127	69.3	104
	22	31.1	46.8	24.4	36.7	17.0	25.6	76.8	115	63.1	94.8
	23	28.5	42.8	22.3	33.5	15.6	23.4	70.3	106	57.7	86.8
	24	26.1	39.3	20.5	30.8	14.3	21.5	64.5	97.0	53.0	79.7
	25	24.1	36.2	18.9	28.4	13.2	19.8	59.5	89.4	48.9	73.4
	26	22.3	33.5	17.5	26.2	12.2	18.3	55.0	82.7	45.2	67.9
	27	20.7	31.0	16.2	24.3	11.3	17.0	51.0	76.7	41.9	63.0
	28			15.1	22.6	10.5	15.8	47.4	71.3	39.0	58.6
	29							44.2	66.4	36.3	54.6
	30							41.3	62.1	33.9	51.0
	32									29.8	44.8

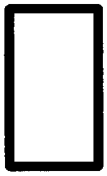
Properties

	HSS8×4×$^{1}/_{4}$	HSS8×4×$^{3}/_{16}$	HSS8×4×$^{1}/_{8}$	HSS7×5×$^{1}/_{2}$	HSS7×5×$^{3}/_{8}$
A_g (in.^{2})	5.24	3.98	2.70	9.74	7.58
I_x (in.^{4})	42.5	33.1	22.9	60.6	49.5
I_y (in.^{4})	14.4	11.3	7.90	35.6	29.3
r_x/r_y	1.72	1.71	1.70	1.30	1.30
r_y (in.)	1.66	1.69	1.71	1.91	1.97

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



HSS7

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

F_y = 46 ksi

Shape		HSS7×5×								HSS7×4×	
		5/16		1/4		3/16^c		1/8^c		1/2	
t_{design}, in.		0.291		0.233		0.174		0.116		0.465	
Wt/ft		23.3		19.0		14.5		9.85		31.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	177	266	144	217	107	161	59.0	88.7	243	365
	6	162	244	132	199	100	151	56.6	85.0	209	314
	7	157	236	128	193	97.7	147	55.7	83.7	198	298
	8	151	228	124	186	94.5	142	54.7	82.1	186	280
	9	145	218	119	179	90.8	136	53.5	80.4	174	261
	10	139	208	114	171	86.9	131	52.1	78.4	161	242
	11	132	198	108	163	82.8	124	50.7	76.1	147	221
	12	125	187	102	154	78.5	118	48.9	73.5	134	201
	13	117	176	96.5	145	74.1	111	46.9	70.5	121	182
	14	110	165	90.5	136	69.6	105	44.7	67.2	108	163
	15	102	154	84.5	127	65.1	97.8	42.5	63.8	96.0	144
	16	94.8	142	78.4	118	60.6	91.0	40.1	60.3	84.4	127
	17	87.4	131	72.5	109	56.1	84.3	37.7	56.6	74.8	112
	18	80.3	121	66.7	100	51.7	77.8	35.2	52.9	66.7	100
	19	73.3	110	61.1	91.9	47.5	71.4	32.6	49.1	59.9	90.0
	20	66.5	100	55.6	83.6	43.4	65.2	30.1	45.2	54.0	81.2
	21	60.4	90.7	50.5	75.9	39.4	59.2	27.4	41.2	49.0	73.7
	22	55.0	82.7	46.0	69.1	35.9	53.9	25.0	37.5	44.7	67.1
	23	50.3	75.6	42.1	63.2	32.8	49.3	22.8	34.3	40.9	61.4
	24	46.2	69.5	38.6	58.1	30.1	45.3	21.0	31.5	37.5	56.4
	25	42.6	64.0	35.6	53.5	27.8	41.8	19.3	29.0	34.6	52.0
	26	39.4	59.2	32.9	49.5	25.7	38.6	17.9	26.8		
	27	36.5	54.9	30.5	45.9	23.8	35.8	16.6	24.9		
	28	33.9	51.0	28.4	42.7	22.1	33.3	15.4	23.2		
	29	31.6	47.6	26.5	39.8	20.6	31.0	14.4	21.6		
	30	29.6	44.4	24.7	37.2	19.3	29.0	13.4	20.2		
	32	26.0	39.1	21.7	32.7	17.0	25.5	11.8	17.7		
	34					15.0	22.6	10.4	15.7		

Properties

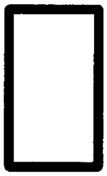
	HSS7×5×5/16	HSS7×5×1/4	HSS7×5×3/16	HSS7×5×1/8	HSS7×4×1/2
A_g (in.^{2})	6.43	5.24	3.98	2.70	8.81
I_x (in.^{4})	43.0	35.9	27.9	19.3	50.7
I_y (in.^{4})	25.5	21.3	16.6	11.6	20.7
r_x/r_y	1.30	1.30	1.29	1.29	1.56
r_y (in.)	1.99	2.02	2.05	2.07	1.53

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS



HSS7

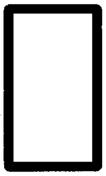
Shape		HSS7×4×									
		3/8		5/16		1/4		3/16^c		1/8^c	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Wt/ft		24.9		21.2		17.3		13.3		9.00	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	189	285	161	242	131	197	97.7	147	55.1	82.9
	6	165	248	141	212	115	173	88.1	132	50.9	76.5
	7	157	236	134	202	110	165	84.2	127	49.4	74.3
	8	148	222	127	191	104	157	79.9	120	47.7	71.8
	9	139	208	119	179	98.0	147	75.3	113	45.9	68.9
	10	129	194	111	167	91.5	138	70.4	106	43.8	65.8
	11	119	179	102	154	84.8	127	65.4	98.3	41.6	62.5
	12	109	163	94.0	141	78.0	117	60.4	90.7	39.2	58.8
	13	98.7	148	85.6	129	71.3	107	55.3	83.1	36.6	55.0
	14	88.9	134	77.4	116	64.6	97.2	50.3	75.6	34.0	51.0
	15	79.5	120	69.5	104	58.2	87.5	45.4	68.3	31.2	46.9
	16	70.4	106	61.8	92.9	52.0	78.2	40.8	61.3	28.3	42.6
	17	62.4	93.7	54.7	82.3	46.1	69.3	36.2	54.5	25.4	38.1
	18	55.6	83.6	48.8	73.4	41.1	61.8	32.3	48.6	22.6	34.0
	19	49.9	75.0	43.8	65.9	36.9	55.5	29.0	43.6	20.3	30.5
	20	45.1	67.7	39.6	59.5	33.3	50.1	26.2	39.3	18.3	27.6
	21	40.9	61.4	35.9	53.9	30.2	45.4	23.7	35.7	16.6	25.0
	22	37.2	56.0	32.7	49.1	27.5	41.4	21.6	32.5	15.2	22.8
	23	34.1	51.2	29.9	45.0	25.2	37.9	19.8	29.8	13.9	20.8
	24	31.3	47.0	27.5	41.3	23.1	34.8	18.2	27.3	12.7	19.1
	25	28.8	43.3	25.3	38.0	21.3	32.1	16.8	25.2	11.7	17.6
	26	26.7	40.1	23.4	35.2	19.7	29.6	15.5	23.3	10.8	16.3
	27					18.3	27.5	14.4	21.6	10.1	15.1
	28									9.35	14.1

Properties

	3/8	5/16	1/4	3/16^c	1/8^c
A_g (in.^{2})	6.88	5.85	4.77	3.63	2.46
I_x (in.^{4})	41.8	36.5	30.5	23.8	16.6
I_y (in.^{4})	17.3	15.2	12.8	10.00	7.03
r_x/r_y	1.56	1.55	1.55	1.54	1.54
r_y (in.)	1.58	1.61	1.64	1.66	1.69

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



HSS6

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

F_y = 46 ksi

Shape		HSS6×5×											
		1/2		3/8		5/16		1/4		3/16		1/8[c]	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Wt/ft		31.7		24.9		21.2		17.3		13.3		9.00	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	243	365	189	285	161	242	131	197	100	150	58.0	87.1
	1	242	364	189	284	161	241	131	197	99.7	150	57.9	87.0
	2	240	361	187	282	159	240	130	195	99.0	149	57.7	86.7
	3	237	356	185	278	157	237	128	193	97.8	147	57.3	86.1
	4	232	349	182	273	155	232	126	190	96.2	145	56.8	85.3
	5	226	340	177	267	151	227	123	186	94.1	141	56.1	84.3
	6	220	330	172	259	147	221	120	181	91.7	138	55.2	83.0
	7	212	318	167	250	142	214	116	175	88.8	134	54.2	81.5
	8	203	305	160	241	137	206	112	169	85.7	129	53.1	79.8
	9	194	291	153	230	131	197	108	162	82.3	124	51.8	77.8
	10	184	276	146	219	125	188	103	154	78.6	118	50.3	75.6
	11	174	261	138	207	118	178	97.4	146	74.7	112	48.6	73.1
	12	163	245	130	195	112	168	92.0	138	70.7	106	46.6	70.0
	13	152	228	122	183	105	157	86.5	130	66.6	100	44.4	66.7
	14	141	212	113	171	97.8	147	80.9	122	62.4	93.7	42.0	63.2
	15	130	196	105	158	90.9	137	75.3	113	58.2	87.4	39.6	59.5
	16	119	180	97.0	146	84.0	126	69.7	105	54.0	81.1	37.1	55.8
	17	109	164	89.0	134	77.2	116	64.3	96.6	49.9	74.9	34.5	51.8
	18	98.9	149	81.2	122	70.6	106	58.9	88.6	45.8	68.9	31.8	47.7
	19	89.1	134	73.7	111	64.3	96.6	53.8	80.8	41.9	63.0	29.1	43.8
	20	80.4	121	66.5	99.9	58.1	87.3	48.7	73.2	38.1	57.3	26.6	40.0
	21	72.9	110	60.3	90.6	52.7	79.2	44.2	66.4	34.6	52.0	24.1	36.2
	22	66.5	99.9	54.9	82.6	48.0	72.2	40.3	60.5	31.5	47.4	22.0	33.0
	23	60.8	91.4	50.3	75.6	43.9	66.0	36.9	55.4	28.8	43.3	20.1	30.2
	24	55.9	83.9	46.2	69.4	40.4	60.7	33.8	50.9	26.5	39.8	18.5	27.7
	25	51.5	77.4	42.6	64.0	37.2	55.9	31.2	46.9	24.4	36.7	17.0	25.6
	26	47.6	71.5	39.3	59.1	34.4	51.7	28.8	43.3	22.6	33.9	15.7	23.6
	27	44.1	66.3	36.5	54.8	31.9	47.9	26.7	40.2	20.9	31.4	14.6	21.9
	28	41.0	61.7	33.9	51.0	29.6	44.6	24.9	37.4	19.4	29.2	13.6	20.4
	29	38.3	57.5	31.6	47.5	27.6	41.5	23.2	34.8	18.1	27.3	12.6	19.0
	30	35.7	53.7	29.5	44.4	25.8	38.8	21.7	32.6	16.9	25.5	11.8	17.8

Properties

A_g (in.^{2})	8.81	6.88	5.85	4.77	3.63	2.46
I_x (in.^{4})	41.1	33.9	29.6	24.7	19.3	13.4
I_y (in.^{4})	30.8	25.5	22.3	18.7	14.6	10.2
r_x/r_y	1.16	1.15	1.15	1.15	1.15	1.15
r_y (in.)	1.87	1.92	1.95	1.98	2.01	2.03

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 46 ksi.

F_y = 46 ksi

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS6

Shape		HSS6×4×											
		1/2		3/8		5/16		1/4		3/16		1/8[c]	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Wt/ft		28.3		22.3		19.1		15.6		12.0		8.15	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	217	326	170	256	145	218	119	178	90.4	136	54.1	81.3
	1	216	325	170	255	144	217	118	177	90.0	135	54.0	81.1
	2	213	321	167	252	143	215	117	175	89.1	134	53.6	80.5
	3	209	314	164	247	140	210	115	172	87.5	131	52.9	79.5
	4	203	305	160	240	136	205	112	168	85.3	128	52.0	78.1
	5	195	293	154	231	132	198	108	162	82.5	124	50.8	76.3
	6	186	279	147	221	126	190	104	156	79.3	119	49.4	74.2
	7	176	264	140	210	120	180	98.6	148	75.7	114	47.7	71.7
	8	165	248	132	198	113	170	93.2	140	71.6	108	45.9	68.9
	9	153	230	123	185	106	159	87.5	131	67.4	101	43.8	65.8
	10	141	212	114	171	98.4	148	81.5	122	62.9	94.5	41.5	62.4
	11	129	194	105	158	90.7	136	75.3	113	58.3	87.6	39.1	58.7
	12	117	176	95.6	144	83.0	125	69.1	104	53.6	80.5	36.5	54.8
	13	105	158	86.5	130	75.3	113	62.9	94.5	48.9	73.6	33.8	50.7
	14	93.6	141	77.6	117	67.8	102	56.8	85.4	44.4	66.7	30.9	46.4
	15	82.5	124	69.1	104	60.6	91.1	51.0	76.6	39.9	60.0	27.9	41.9
	16	72.5	109	60.9	91.5	53.6	80.6	45.3	68.1	35.7	53.6	25.0	37.6
	17	64.2	96.5	53.9	81.1	47.5	71.4	40.1	60.3	31.6	47.5	22.2	33.4
	18	57.3	86.1	48.1	72.3	42.4	63.7	35.8	53.8	28.2	42.4	19.8	29.8
	19	51.4	77.3	43.2	64.9	38.0	57.2	32.1	48.3	25.3	38.1	17.8	26.7
	20	46.4	69.8	39.0	58.6	34.3	51.6	29.0	43.6	22.9	34.4	16.0	24.1
	21	42.1	63.3	35.3	53.1	31.1	46.8	26.3	39.5	20.7	31.2	14.6	21.9
	22	38.4	57.6	32.2	48.4	28.4	42.6	24.0	36.0	18.9	28.4	13.3	19.9
	23	35.1	52.7	29.5	44.3	26.0	39.0	21.9	33.0	17.3	26.0	12.1	18.2
	24	32.2	48.4	27.1	40.7	23.8	35.8	20.1	30.3	15.9	23.9	11.1	16.8
	25	29.7	44.6	24.9	37.5	22.0	33.0	18.6	27.9	14.6	22.0	10.3	15.4
	26					20.3	30.5	17.2	25.8	13.5	20.3	9.50	14.3
	27									12.5	18.8	8.81	13.2

Properties

	1/2	3/8	5/16	1/4	3/16	1/8
A_g (in.^{2})	7.88	6.18	5.26	4.30	3.28	2.23
I_x (in.^{4})	34.0	28.3	24.8	20.9	16.4	11.4
I_y (in.^{4})	17.8	14.9	13.2	11.1	8.76	6.15
r_x/r_y	1.38	1.38	1.37	1.37	1.37	1.36
r_y (in.)	1.50	1.55	1.58	1.61	1.63	1.66

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

Table 4-3 (continued)
Available Strength in Axial Compression, kips

F_y = 46 ksi

Rectangular HSS

HSS6

Shape		HSS6×3×											
		1/2		3/8		5/16		1/4		3/16		1/8^c	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Wt/ft		24.9		19.7		16.9		13.9		10.7		8.15	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	191	288	151	227	129	194	106	159	80.8	121	47.7	71.7
	1	190	285	150	225	128	192	105	158	80.3	121	47.5	71.4
	2	186	279	147	221	125	189	103	155	78.8	118	46.9	70.5
	3	178	268	142	213	121	182	99.7	150	76.4	115	46.0	69.1
	4	169	254	135	203	116	174	95.2	143	73.1	110	44.7	67.1
	5	158	237	126	190	109	163	89.8	135	69.1	104	43.0	64.6
	6	145	218	117	176	101	152	83.6	126	64.5	97.0	41.0	61.6
	7	131	197	107	160	92.4	139	76.8	115	59.5	89.4	38.7	58.1
	8	117	175	95.9	144	83.4	125	69.6	105	54.2	81.4	36.1	54.2
	9	102	154	85.0	128	74.3	112	62.3	93.7	48.7	73.2	33.2	49.9
	10	88.2	133	74.2	112	65.3	98.2	55.1	82.8	43.3	65.0	30.1	45.2
	11	75.0	113	64.0	96.1	56.6	85.1	48.0	72.2	37.9	57.0	26.7	40.1
	12	63.0	94.7	54.2	81.5	48.3	72.7	41.3	62.1	32.9	49.4	23.2	34.9
	13	53.7	80.6	46.2	69.4	41.2	61.9	35.2	52.9	28.1	42.2	20.0	30.0
	14	46.3	69.5	39.8	59.9	35.5	53.4	30.4	45.6	24.2	36.4	17.2	25.9
	15	40.3	60.6	34.7	52.2	30.9	46.5	26.5	39.8	21.1	31.7	15.0	22.5
	16	35.4	53.2	30.5	45.8	27.2	40.9	23.3	34.9	18.5	27.9	13.2	19.8
	17	31.4	47.2	27.0	40.6	24.1	36.2	20.6	31.0	16.4	24.7	11.7	17.5
	18	28.0	42.1	24.1	36.2	21.5	32.3	18.4	27.6	14.7	22.0	10.4	15.6
	19			21.6	32.5	19.3	29.0	16.5	24.8	13.2	19.8	9.34	14.0
	20							14.9	22.4	11.9	17.8	8.43	12.7
	21											7.65	11.5

Properties

	1/2	3/8	5/16	1/4	3/16	1/8^c
A_g (in.^{2})	6.95	5.48	4.68	3.84	2.93	2.00
I_x (in.^{4})	26.8	22.7	20.1	17.0	13.4	9.43
I_y (in.^{4})	8.69	7.48	6.67	5.70	4.55	3.23
r_x/r_y	1.76	1.74	1.73	1.73	1.72	1.71
r_y (in.)	1.12	1.17	1.19	1.22	1.25	1.27

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS5

Shape		HSS5×4×											
		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$[c]	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Wt/ft		24.9		19.7		16.9		13.9		10.7		7.30	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	191	288	151	227	129	194	106	159	80.8	121	52.6	79.0
	1	191	286	150	226	128	193	105	158	80.5	121	52.4	78.8
	2	188	282	148	223	127	191	104	156	79.6	120	52.0	78.1
	3	184	276	145	218	124	187	102	153	78.1	117	51.2	77.0
	4	178	268	141	212	121	182	99.2	149	76.0	114	50.2	75.4
	5	171	257	136	204	116	175	95.8	144	73.5	110	48.9	73.4
	6	163	244	130	195	111	167	91.7	138	70.5	106	47.3	71.1
	7	153	230	123	185	106	159	87.2	131	67.1	101	45.4	68.3
	8	143	215	115	173	99.4	149	82.2	124	63.4	95.2	43.4	65.2
	9	133	199	107	161	92.7	139	76.9	116	59.4	89.3	40.9	61.5
	10	122	183	99.1	149	85.8	129	71.3	107	55.3	83.1	38.2	57.3
	11	111	166	90.7	136	78.8	118	65.7	98.7	51.0	76.7	35.3	53.1
	12	99.7	150	82.3	124	71.8	108	60.0	90.2	46.8	70.3	32.5	48.8
	13	89.0	134	74.1	111	64.8	97.5	54.4	81.8	42.5	63.9	29.6	44.5
	14	78.8	118	66.1	99.4	58.1	87.3	48.9	73.5	38.4	57.7	26.8	40.3
	15	68.9	104	58.5	87.9	51.6	77.6	43.7	65.6	34.4	51.7	24.1	36.2
	16	60.6	91.1	51.4	77.2	45.5	68.3	38.6	58.0	30.5	45.9	21.5	32.3
	17	53.7	80.7	45.5	68.4	40.3	60.5	34.2	51.3	27.0	40.6	19.1	28.6
	18	47.9	72.0	40.6	61.0	35.9	54.0	30.5	45.8	24.1	36.2	17.0	25.5
	19	43.0	64.6	36.4	54.8	32.2	48.4	27.3	41.1	21.6	32.5	15.3	22.9
	20	38.8	58.3	32.9	49.4	29.1	43.7	24.7	37.1	19.5	29.4	13.8	20.7
	21	35.2	52.9	29.8	44.8	26.4	39.7	22.4	33.6	17.7	26.6	12.5	18.8
	22	32.1	48.2	27.2	40.8	24.0	36.1	20.4	30.7	16.1	24.3	11.4	17.1
	23	29.3	44.1	24.9	37.4	22.0	33.1	18.7	28.1	14.8	22.2	10.4	15.6
	24	26.9	40.5	22.8	34.3	20.2	30.4	17.1	25.8	13.6	20.4	9.56	14.4
	25			21.0	31.6	18.6	28.0	15.8	23.7	12.5	18.8	8.81	13.2
	26							14.6	22.0	11.6	17.4	8.14	12.2
	27											7.55	11.4

Properties

	$^1/_2$	$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$	$^1/_8$
A_g (in.^{2})	6.95	5.48	4.68	3.84	2.93	2.00
I_x (in.^{4})	21.2	17.9	15.8	13.4	10.6	7.42
I_y (in.^{4})	14.9	12.6	11.1	9.46	7.48	5.27
r_x/r_y	1.19	1.19	1.19	1.19	1.19	1.19
r_y (in.)	1.46	1.52	1.54	1.57	1.60	1.62

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

HSS5

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

F_y = 46 ksi

Shape		HSS5×3×											
		1/2		3/8		5/16		1/4		3/16		1/8^c	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Wt/ft		21.5		17.2		14.8		12.2		9.43		6.45	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	166	249	132	198	113	170	92.9	140	71.2	107	46.2	69.4
	1	164	247	131	197	112	169	92.2	139	70.7	106	46.0	69.1
	2	160	241	128	192	110	165	90.4	136	69.4	104	45.3	68.1
	3	154	232	123	185	106	159	87.3	131	67.2	101	44.2	66.5
	4	146	219	117	176	101	151	83.3	125	64.2	96.4	42.8	64.3
	5	135	203	109	164	94.6	142	78.4	118	60.5	91.0	40.9	61.5
	6	124	186	101	152	87.5	131	72.7	109	56.3	84.7	38.6	58.1
	7	111	167	91.6	138	79.8	120	66.6	100	51.8	77.8	35.9	53.9
	8	98.6	148	82.0	123	71.7	108	60.1	90.4	47.0	70.6	32.7	49.1
	9	85.9	129	72.2	109	63.6	95.5	53.6	80.5	42.1	63.2	29.4	44.2
	10	73.6	111	62.7	94.3	55.5	83.5	47.1	70.8	37.2	55.9	26.1	39.2
	11	61.9	93.1	53.7	80.7	47.9	71.9	40.8	61.4	32.4	48.7	22.9	34.4
	12	52.0	78.2	45.3	68.0	40.6	61.0	34.9	52.4	27.9	42.0	19.8	29.8
	13	44.3	66.6	38.6	58.0	34.6	52.0	29.7	44.6	23.8	35.8	17.0	25.5
	14	38.2	57.4	33.3	50.0	29.8	44.8	25.6	38.5	20.5	30.8	14.6	22.0
	15	33.3	50.0	29.0	43.6	26.0	39.0	22.3	33.5	17.9	26.9	12.7	19.2
	16	29.3	44.0	25.5	38.3	22.8	34.3	19.6	29.5	15.7	23.6	11.2	16.8
	17	25.9	39.0	22.6	33.9	20.2	30.4	17.4	26.1	13.9	20.9	9.92	14.9
	18	23.1	34.8	20.1	30.2	18.0	27.1	15.5	23.3	12.4	18.7	8.85	13.3
	19			18.1	27.1	16.2	24.3	13.9	20.9	11.1	16.7	7.95	11.9
	20									10.1	15.1	7.17	10.8

Properties

	1/2	3/8	5/16	1/4	3/16	1/8^c
A_g (in.^{2})	6.02	4.78	4.10	3.37	2.58	1.77
I_x (in.^{4})	16.4	14.1	12.6	10.7	8.53	6.03
I_y (in.^{4})	7.18	6.25	5.60	4.81	3.85	2.75
r_x/r_y	1.51	1.50	1.50	1.49	1.49	1.48
r_y (in.)	1.09	1.14	1.17	1.19	1.22	1.25

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS5–HSS4

Shape		HSS5×2½×						HSS4×3×					
		¼		$^3/_{16}$		$^1/_8$^{c}		⅜		$^5/_{16}$		¼	
t_{design}, in.		0.233		0.174		0.116		0.349		0.291		0.233	
Wt/ft		11.3		8.79		6.02		14.6		12.7		10.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	86.4	130	66.4	99.8	43.0	64.6	113	169	96.9	146	80.0	120
	1	85.6	129	65.8	98.9	42.7	64.2	112	168	96.2	145	79.4	119
	2	83.1	125	64.0	96.2	41.9	63.0	109	164	94.0	141	77.7	117
	3	79.2	119	61.1	91.9	40.5	60.9	105	158	90.5	136	75.0	113
	4	74.0	111	57.3	86.1	38.6	58.0	99.2	149	85.9	129	71.3	107
	5	67.8	102	52.7	79.3	36.2	54.5	92.4	139	80.3	121	66.9	100
	6	60.9	91.6	47.6	71.6	33.2	49.9	84.7	127	73.9	111	61.8	92.8
	7	53.7	80.7	42.3	63.5	29.6	44.5	76.4	115	67.0	101	56.3	84.6
	8	46.4	69.8	36.8	55.3	25.9	39.0	67.9	102	59.8	89.9	50.5	75.9
	9	39.4	59.2	31.5	47.3	22.3	33.6	59.4	89.2	52.6	79.1	44.7	67.2
	10	32.7	49.1	26.4	39.7	18.9	28.4	51.1	76.8	45.6	68.6	39.0	58.6
	11	27.0	40.6	21.8	32.8	15.7	23.6	43.2	65.0	39.0	58.5	33.5	50.4
	12	22.7	34.1	18.4	27.6	13.2	19.9	36.3	54.6	32.8	49.3	28.4	42.6
	13	19.3	29.1	15.6	23.5	11.3	16.9	31.0	46.5	27.9	42.0	24.2	36.3
	14	16.7	25.1	13.5	20.3	9.71	14.6	26.7	40.1	24.1	36.2	20.8	31.3
	15	14.5	21.8	11.7	17.7	8.46	12.7	23.3	35.0	21.0	31.5	18.2	27.3
	16	12.8	19.2	10.3	15.5	7.43	11.2	20.4	30.7	18.4	27.7	16.0	24.0
	17			9.15	13.7	6.58	9.90	18.1	27.2	16.3	24.6	14.1	21.3
	18							16.1	24.3	14.6	21.9	12.6	19.0
	19											11.3	17.0

Properties

	HSS5×2½×¼	HSS5×2½×$^3/_{16}$	HSS5×2½×$^1/_8$	HSS4×3×⅜	HSS4×3×$^5/_{16}$	HSS4×3×¼
A_g (in.^{2})	3.14	2.41	1.65	4.09	3.52	2.91
I_x (in.^{4})	9.40	7.51	5.34	7.93	7.14	6.15
I_y (in.^{4})	3.13	2.53	1.82	5.01	4.52	3.91
r_x/r_y	1.73	1.72	1.71	1.26	1.26	1.25
r_y (in.)	0.999	1.02	1.05	1.11	1.13	1.16

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

Table 4–3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS4

F_y = 46 ksi

Shape	HSS4×3×				HSS4×2$^1/_2$×							
	$^3/_{16}$		$^1/_8$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$	
t_{design}, in.	0.174		0.116		0.349		0.291		0.233		0.174	
Wt/ft	8.15		5.60		13.4		11.6		9.63		7.51	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y 0	61.6	92.6	42.3	63.6	103	155	88.9	134	73.6	111	56.8	85.4
1	61.2	92.0	42.0	63.2	102	153	87.9	132	72.9	109	56.3	84.6
2	59.9	90.1	41.2	61.9	98.3	148	85.1	128	70.6	106	54.7	82.2
3	57.9	87.1	39.9	59.9	92.9	140	80.6	121	67.1	101	52.1	78.3
4	55.2	83.0	38.1	57.2	85.7	129	74.8	112	62.5	93.9	48.7	73.1
5	51.9	78.0	35.9	53.9	77.4	116	67.8	102	57.0	85.6	44.6	67.0
6	48.1	72.3	33.4	50.2	68.3	103	60.2	90.5	50.9	76.5	40.1	60.2
7	44.0	66.2	30.7	46.1	58.8	88.4	52.4	78.7	44.6	67.0	35.3	53.1
8	39.7	59.7	27.8	41.8	49.6	74.5	44.5	66.9	38.2	57.5	30.5	45.9
9	35.3	53.1	24.8	37.3	40.9	61.4	37.1	55.7	32.1	48.3	25.9	38.9
10	31.0	46.6	21.9	33.0	33.1	49.8	30.2	45.4	26.4	39.7	21.5	32.3
11	26.9	40.4	19.1	28.7	27.4	41.1	25.0	37.5	21.8	32.8	17.8	26.7
12	22.9	34.4	16.4	24.7	23.0	34.6	21.0	31.5	18.3	27.6	14.9	22.5
13	19.5	29.3	14.0	21.0	19.6	29.5	17.9	26.9	15.6	23.5	12.7	19.1
14	16.8	25.3	12.1	18.1	16.9	25.4	15.4	23.2	13.5	20.2	11.0	16.5
15	14.6	22.0	10.5	15.8	14.7	22.1	13.4	20.2	11.7	17.6	9.56	14.4
16	12.9	19.3	9.24	13.9					10.3	15.5	8.40	12.6
17	11.4	17.1	8.18	12.3								
18	10.2	15.3	7.30	11.0								
19	9.13	13.7	6.55	9.84								
20			5.91	8.88								

Properties

	HSS4×3×$^3/_{16}$	HSS4×3×$^1/_8$	HSS4×2$^1/_2$×$^3/_8$	HSS4×2$^1/_2$×$^5/_{16}$	HSS4×2$^1/_2$×$^1/_4$	HSS4×2$^1/_2$×$^3/_{16}$
A_g (in.^{2})	2.24	1.54	3.74	3.23	2.67	2.06
I_x (in.^{4})	4.93	3.52	6.77	6.13	5.32	4.30
I_y (in.^{4})	3.16	2.27	3.17	2.89	2.53	2.06
r_x/r_y	1.25	1.25	1.46	1.46	1.45	1.44
r_y (in.)	1.19	1.21	0.922	0.947	0.973	0.999

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi

Table 4-3 (continued)
Available Strength in Axial Compression, kips
Rectangular HSS

HSS4

Shape		HSS4×2½×		HSS4×2×									
		1/8		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.116		0.349		0.291		0.233		0.174		0.116	
Wt/ft		5.17		12.1		10.5		8.78		6.87		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	39.1	58.8	93.3	140	80.8	122	67.2	101	52.0	78.2	35.9	54.0
	1	38.7	58.2	91.6	138	79.5	119	66.1	99.4	51.3	77.0	35.4	53.2
	2	37.7	56.6	86.7	130	75.5	113	63.0	94.7	49.0	73.7	33.9	51.0
	3	36.0	54.1	79.2	119	69.3	104	58.2	87.4	45.5	68.3	31.6	47.5
	4	33.7	50.7	69.7	105	61.5	92.5	52.0	78.2	40.9	61.5	28.7	43.1
	5	31.1	46.7	59.1	88.9	52.8	79.3	45.1	67.7	35.8	53.8	25.3	38.0
	6	28.1	42.2	48.4	72.8	43.8	65.8	37.8	56.8	30.4	45.6	21.6	32.5
	7	24.9	37.4	38.2	57.4	35.0	52.7	30.7	46.2	25.0	37.6	18.0	27.1
	8	21.7	32.6	29.4	44.1	27.2	40.9	24.1	36.2	19.9	30.0	14.6	22.0
	9	18.5	27.9	23.2	34.9	21.5	32.3	19.1	28.6	15.8	23.7	11.6	17.4
	10	15.6	23.4	18.8	28.2	17.4	26.1	15.4	23.2	12.8	19.2	9.38	14.1
	11	12.9	19.4	15.5	23.3	14.4	21.6	12.8	19.2	10.5	15.9	7.75	11.6
	12	10.8	16.3	13.0	19.6	12.1	18.2	10.7	16.1	8.86	13.3	6.51	9.79
	13	9.22	13.9							7.55	11.3	5.55	8.34
	14	7.95	12.0										
	15	6.93	10.4										
	16	6.09	9.15										
	17	5.39	8.11										

Properties

	HSS4×2½×1/8	HSS4×2×3/8	HSS4×2×5/16	HSS4×2×1/4	HSS4×2×3/16	HSS4×2×1/8
A_g (in.^{2})	1.42	3.39	2.94	2.44	1.89	1.30
I_x (in.^{4})	3.09	5.60	5.13	4.49	3.66	2.65
I_y (in.^{4})	1.49	1.80	1.67	1.48	1.22	0.898
r_x/r_y	1.44	1.76	1.75	1.74	1.73	1.72
r_y (in.)	1.03	0.729	0.754	0.779	0.804	0.830

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.

Table 4-4
Available Strength in Axial Compression, kips
Square HSS

HSS16–HSS14

$F_y = 46$ ksi

Shape		HSS16×16×						HSS14×14×					
		1/2		3/8^c		5/16^c		5/8		1/2		3/8^c	
t_{design}, in.		0.465		0.349		0.291		0.581		0.465		0.349	
Wt/ft		103		78.4		65.8		110		89.6		68.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	781	1170	521	783	380	571	835	1260	678	1020	499	750
	6	774	1160	518	779	378	569	825	1240	670	1010	495	744
	7	771	1160	518	778	378	568	822	1240	668	1000	494	742
	8	769	1160	516	776	377	567	818	1230	664	998	492	740
	9	765	1150	515	774	376	566	813	1220	661	993	490	737
	10	762	1150	514	772	376	564	808	1210	657	987	488	734
	11	758	1140	512	770	375	563	803	1210	652	980	486	731
	12	754	1130	511	768	374	562	797	1200	647	973	484	727
	13	749	1130	509	765	373	560	790	1190	642	965	481	723
	14	744	1120	507	762	371	558	783	1180	637	957	478	719
	15	739	1110	505	759	370	556	776	1170	631	948	475	715
	16	733	1100	503	755	369	554	768	1150	625	939	472	710
	17	728	1090	500	752	367	552	760	1140	618	929	469	704
	18	721	1080	498	748	366	550	751	1130	611	919	465	699
	19	715	1070	495	744	364	547	742	1120	604	908	461	693
	20	708	1060	492	740	363	545	733	1100	596	896	455	684
	21	701	1050	489	735	361	542	723	1090	589	885	449	675
	22	694	1040	486	731	359	539	713	1070	581	873	443	666
	23	686	1030	483	726	357	536	702	1060	572	860	437	657
	24	679	1020	479	720	355	533	691	1040	564	847	431	647
	25	670	1010	476	715	353	530	680	1020	555	834	424	637
	26	662	995	472	709	350	527	669	1010	546	820	417	627
	27	654	983	468	704	348	523	658	988	537	807	410	617
	28	645	970	464	698	346	520	646	971	527	792	403	606
	29	636	956	460	691	343	516	634	953	518	778	396	596
	30	627	943	456	685	340	512	622	935	508	764	389	585
	32	608	915	446	671	335	503	597	897	488	734	374	562
	34	589	886	437	656	329	494	572	859	468	703	359	539
	36	570	856	426	640	323	485	546	821	447	672	343	516
	38	549	826	415	624	316	475	520	782	427	641	328	493
	40	529	795	403	606	309	464	494	743	406	610	312	469

Properties

A_g (in.^{2})	28.3	21.5	18.1	30.3	24.6	18.7
$I_x = I_y$ (in.^{4})	1130	873	739	897	743	577
$r_x = r_y$ (in.^{4})	6.31	6.37	6.39	5.44	5.49	5.55

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with $F_y = 46$ ksi.

F_y = 46 ksi

Table 4–4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS14–HSS12

Shape		HSS14×14×		HSS12×12×									
		$^5/_{16}{}^c$		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}{}^c$		$^1/_4{}^c$	
t_{design}, in.		0.291		0.581		0.465		0.349		0.291		0.233	
Wt/ft		57.3		93.1		75.9		58.0		48.8		39.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	367	552	707	1060	576	865	439	660	350	526	240	
	6	365	549	696	1050	567	852	433	650	347	521	238	
	7	364	547	691	1040	563	847	430	647	345	519	237	
	8	363	546	687	1030	560	841	427	642	344	517	236	
	9	362	544	682	1020	555	835	424	638	342	515	236	
	10	361	543	676	1020	551	828	421	632	341	512	235	
	11	360	541	669	1010	546	820	417	627	339	509	233	
	12	358	538	662	995	540	812	413	621	336	505	232	
	13	357	536	655	984	534	803	408	614	334	502	231	
	14	355	534	647	972	528	793	404	607	331	498	230	
	15	353	531	638	960	521	783	399	599	328	494	228	
	16	351	528	630	946	514	772	393	591	325	489	226	
	17	349	525	620	932	506	761	388	583	322	484	225	
	18	347	521	610	917	499	750	382	574	319	479	223	
	19	344	518	600	902	491	737	376	565	315	474	221	
	20	342	514	590	886	482	725	370	555	311	468	219	
	21	339	510	579	870	474	712	363	546	306	460	217	
	22	337	506	568	853	465	698	356	536	300	451	214	
	23	334	502	556	836	455	685	350	525	295	443	212	
	24	331	497	544	818	446	670	342	515	289	434	209	
	25	328	492	533	800	436	656	335	504	283	425	207	
	26	324	487	520	782	427	641	328	493	277	416	204	
	27	321	482	508	763	417	626	321	482	271	407	201	
	28	317	477	495	745	407	611	313	470	264	397	198	
	29	314	471	483	726	397	596	305	459	258	388	195	
	30	310	465	470	706	386	581	298	447	251	378	192	
	32	302	453	444	668	366	550	282	424	238	358	185	
	34	293	440	419	629	345	519	267	401	225	339	177	
	36	284	426	393	590	324	487	251	377	212	319	170	
	38	274	411	367	552	304	456	235	354	199	300	161	
	40	263	396	342	514	283	426	220	331	186	280	152	

Properties

	HSS14×14×$^5/_{16}$	HSS12×12×$^5/_8$	$^1/_2$	$^3/_8$	$^5/_{16}$	$^1/_4$
A_g (in.^{2})	15.7	25.7	20.9	16.0	13.4	10.8
$I_x = I_y$ (in.^{4})	490	548	457	357	304	248
$r_x = r_y$ (in.^{4})	5.58	4.62	4.68	4.73	4.76	4.79

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Table 4–4 (continued)
Available Strength in Axial Compression, kips

HSS12–HSS10

Square HSS

F_y = 46 ksi

Shape		HSS12×12×		HSS10×10×									
		$^3/_{16}{}^c$		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4{}^c$	
t_{design}, in.		0.174		0.581		0.465		0.349		0.291		0.233	
Wt/ft		29.8		76.1		62.3		47.8		40.3		32.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	142	214	579	870	473	711	362	545	305	459	228	342
	6	141	212	565	850	462	695	354	533	299	449	224	337
	7	141	212	560	842	458	689	351	528	296	445	223	336
	8	141	211	555	834	454	682	348	523	293	441	222	334
	9	140	211	548	824	449	675	344	518	290	436	221	331
	10	140	210	542	814	443	666	340	511	287	431	219	329
	11	139	209	534	803	437	657	336	505	283	426	217	326
	12	139	209	526	790	431	648	331	497	279	419	215	323
	13	138	208	517	777	424	637	326	490	275	413	213	320
	14	138	207	508	763	417	626	320	481	270	406	211	316
	15	137	206	498	749	409	614	314	473	265	399	208	313
	16	136	205	488	733	401	602	308	463	260	391	205	309
	17	136	204	477	717	392	589	302	454	255	383	202	304
	18	135	202	466	701	383	576	295	444	250	375	199	300
	19	134	201	455	683	374	562	289	434	244	366	196	295
	20	133	200	443	666	365	548	282	423	238	358	193	289
	21	132	198	431	648	355	534	274	412	232	349	188	283
	22	131	197	419	629	345	519	267	401	226	339	183	276
	23	130	195	406	611	335	504	260	390	220	330	178	268
	24	129	194	394	592	325	489	252	379	213	321	173	261
	25	128	192	381	573	315	474	244	367	207	311	168	253
	26	127	190	368	554	305	458	236	355	200	301	163	245
	27	125	188	355	534	295	443	229	344	194	291	158	237
	28	124	186	343	515	284	427	221	332	187	282	153	229
	29	123	184	330	496	274	412	213	320	181	272	147	221
	30	121	182	317	476	264	396	205	309	174	262	142	214
	32	118	178	292	439	243	366	190	285	161	242	132	198
	34	115	173	267	401	223	335	175	262	149	223	121	182
	36	111	167	243	366	204	306	160	240	136	205	111	167
	38	108	162	220	331	185	278	146	219	124	187	102	153
	40	104	156	198	298	167	251	132	198	112	169	92.2	139

Properties

A_g (in.^{2})	8.15	21.0	17.2	13.2	11.1	8.96
$I_x = I_y$ (in.^{4})	189	304	256	202	172	141
$r_x = r_y$ (in.^{4})	4.82	3.80	3.86	3.92	3.94	3.97

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 46 ksi.

F_y = 46 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS10–HSS9

Shape		HSS10×10×		HSS9×9×									
		$^3/_{16}{}^c$		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4{}^c$	
t_{design}, in.		0.174		0.581		0.465		0.349		0.291		0.233	
Wt/ft		24.7		67.6		55.5		42.7		36.0		29.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	137	206	515	774	422	634	324	487	273	411	219	330
	6	136	204	500	751	410	616	315	473	266	399	215	323
	7	135	203	494	743	405	609	312	469	263	395	213	320
	8	135	203	488	734	401	602	308	463	260	391	211	317
	9	134	202	481	723	395	594	304	457	257	386	208	313
	10	133	200	473	712	389	585	299	450	253	380	205	308
	11	133	199	465	699	382	575	295	443	249	374	202	303
	12	132	198	456	686	375	564	289	435	244	367	198	298
	13	131	196	447	672	368	553	284	426	240	360	194	292
	14	130	195	437	657	360	541	278	417	235	353	190	286
	15	129	193	426	641	351	528	271	408	230	345	186	280
	16	127	191	415	624	343	515	265	398	224	337	182	273
	17	126	189	404	607	334	501	258	388	218	328	177	267
	18	125	187	392	590	324	487	251	377	213	319	173	260
	19	123	185	380	572	315	473	244	367	207	310	168	252
	20	122	183	368	553	305	458	236	355	200	301	163	245
	21	120	180	356	534	295	443	229	344	194	292	158	237
	22	118	178	343	515	285	428	221	333	188	282	153	230
	23	117	175	330	496	274	413	214	321	181	273	148	222
	24	115	172	317	477	264	397	206	309	175	263	143	214
	25	113	169	305	458	254	382	198	298	168	253	137	206
	26	111	166	292	439	244	366	190	286	162	243	132	199
	27	108	163	279	420	233	351	183	274	155	233	127	191
	28	106	159	267	401	223	335	175	263	149	224	122	183
	29	104	156	254	382	213	320	167	251	142	214	116	175
	30	101	152	242	363	203	305	160	240	136	204	111	167
	32	96.0	144	218	327	184	276	145	217	124	186	101	152
	34	90.3	136	195	293	165	248	130	196	112	168	91.6	138
	36	84.2	127	174	261	147	221	117	175	99.9	150	82.1	123
	38	77.7	117	156	234	132	198	105	157	89.6	135	73.7	111
	40	70.6	106	141	211	119	179	94.4	142	80.9	122	66.5	100

Properties

A_g (in.^{2})	6.76	18.7	15.3	11.8	9.92	8.03
$I_x = I_y$ (in.^{4})	108	216	183	145	124	102
$r_x = r_y$ (in.^{4})	4.00	3.40	3.45	3.51	3.54	3.56

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS9–HSS8

$F_y = 46$ ksi

Shape		HSS9×9×				HSS8×8×							
		$^3/_{16}{}^c$		$^1/_8{}^c$		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$	
t_{design}, in.		0.174		0.116		0.581		0.465		0.349		0.291	
Wt/ft		22.2		15.0		59.1		48.7		37.6		31.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	134	201	64.3	96.7	451	678	371	557	286	429	241	362
	6	132	198	63.8	95.8	434	652	357	537	275	414	233	350
	7	131	197	63.6	95.5	428	643	352	529	272	409	230	345
	8	131	196	63.3	95.2	421	632	347	521	268	402	226	340
	9	130	195	63.0	94.8	413	621	341	512	263	396	223	335
	10	129	193	62.7	94.3	405	608	334	502	258	388	218	328
	11	128	192	62.4	93.8	395	594	327	491	253	380	214	322
	12	126	190	62.0	93.2	386	580	319	479	247	371	209	314
	13	125	188	61.6	92.6	375	564	311	467	241	362	204	307
	14	124	186	61.2	91.9	365	548	302	454	234	352	199	299
	15	122	184	60.7	91.2	353	531	293	440	228	342	193	290
	16	121	181	60.2	90.5	342	513	284	426	221	332	187	281
	17	119	179	59.6	89.6	330	495	274	412	213	321	181	272
	18	117	176	59.1	88.8	317	477	264	397	206	310	175	263
	19	115	173	58.4	87.8	305	458	254	382	198	298	169	253
	20	113	170	57.8	86.8	292	439	244	367	191	287	162	244
	21	111	166	57.1	85.8	279	420	234	351	183	275	156	234
	22	108	163	56.4	84.7	267	401	223	336	175	263	149	224
	23	106	159	55.6	83.5	254	382	213	320	168	252	143	215
	24	103	155	54.8	82.3	241	363	203	305	160	240	136	205
	25	101	151	53.9	81.0	229	344	193	290	152	229	130	195
	26	97.8	147	53.0	79.7	216	325	183	275	144	217	123	186
	27	94.8	143	52.1	78.3	204	307	173	260	137	206	117	176
	28	91.7	138	51.1	76.8	193	289	163	245	130	195	111	167
	29	88.5	133	50.1	75.2	181	272	154	231	122	184	105	158
	30	85.0	128	49.0	73.6	169	255	145	217	115	173	98.9	149
	32	77.4	116	46.7	70.1	149	224	127	191	102	153	87.3	131
	34	70.1	105	44.1	66.3	132	198	113	169	89.9	135	77.3	116
	36	63.0	94.7	41.4	62.2	118	177	100	151	80.2	121	69.0	104
	38	56.5	85.0	38.3	57.6	106	159	90.1	135	72.0	108	61.9	93.0
	40	51.0	76.7	34.9	52.5	95.3	143	81.3	122	65.0	97.6	55.9	83.9

Properties

	HSS9×9×$^3/_{16}$	HSS9×9×$^1/_8$	HSS8×8×$^5/_8$	HSS8×8×$^1/_2$	HSS8×8×$^3/_8$	HSS8×8×$^5/_{16}$
A_g (in.^{2})	6.06	4.09	16.4	13.5	10.4	8.76
$I_x = I_y$ (in.^{4})	78.2	53.5	146	125	100	85.6
$r_x = r_y$ (in.^{4})	3.59	3.62	2.99	3.04	3.10	3.13

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with $F_y = 46$ ksi.

F_y = 46 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS8–HSS7

Shape		HSS8×8×						HSS7×7×					
		1/4		3/16^c		1/8^c		5/8		1/2		3/8	
t_{design}, in.		0.233		0.174		0.116		0.581		0.465		0.349	
Wt/ft		25.8		19.6		13.3		50.6		41.9		32.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	196	294	130	195	63.1	94.9	387	582	319	480	247	
	6	189	284	127	191	62.3	93.7	367	552	304	457	235	
	7	186	280	126	190	62.0	93.2	360	542	298	448	231	
	8	184	276	125	188	61.7	92.7	353	530	292	439	227	
	9	181	272	124	186	61.3	92.2	344	517	285	429	222	
	10	177	267	122	184	60.9	91.5	334	503	278	418	216	
	11	174	261	121	182	60.4	90.8	324	488	270	406	210	
	12	170	255	119	179	59.9	90.0	314	472	261	393	204	
	13	166	249	117	177	59.3	89.1	303	455	252	379	197	
	14	162	243	115	174	58.7	88.2	291	437	243	365	190	
	15	157	236	113	170	58.0	87.2	279	419	233	351	183	
	16	152	229	111	167	57.3	86.1	266	401	224	336	175	
	17	148	222	109	163	56.5	84.9	254	382	213	321	168	
	18	143	214	106	159	55.7	83.7	241	363	203	306	160	
	19	138	207	103	155	54.8	82.3	229	344	193	290	152	
	20	132	199	100	151	53.8	80.9	216	325	183	275	145	
	21	127	191	97.0	146	52.8	79.4	204	306	173	260	137	
	22	122	184	93.1	140	51.8	77.8	191	287	163	244	129	
	23	117	176	89.2	134	50.7	76.2	179	269	153	230	122	
	24	112	168	85.3	128	49.5	74.4	167	251	143	215	114	
	25	106	160	81.4	122	48.3	72.5	156	234	134	201	107	
	26	101	152	77.5	116	47.0	70.6	144	217	124	187	100	
	27	96.2	145	73.7	111	45.6	68.5	134	201	115	173	93.0	
	28	91.2	137	69.9	105	44.1	66.3	124	187	107	161	86.5	
	29	86.3	130	66.2	99.5	42.6	64.1	116	174	99.9	150	80.6	
	30	81.4	122	62.6	94.0	41.0	61.7	108	163	93.4	140	75.3	
	32	72.0	108	55.5	83.3	37.6	56.5	95.2	143	82.1	123	66.2	
	34	63.8	95.9	49.1	73.8	33.7	50.7	84.3	127	72.7	109	58.7	
	36	56.9	85.5	43.8	65.9	30.1	45.2	75.2	113	64.8	97.5	52.3	
	38	51.1	76.8	39.3	59.1	27.0	40.6	67.5	101	58.2	87.5	47.0	
	40	46.1	69.3	35.5	53.3	24.4	36.6	60.9	91.6	52.5	78.9	42.4	

Properties

	HSS8×8×1/4	HSS8×8×3/16^c	HSS8×8×1/8^c	HSS7×7×5/8	HSS7×7×1/2	HSS7×7×3/8
A_g (in.^{2})	7.10	5.37	3.62	14.0	11.6	8.97
$I_x = I_y$ (in.^{4})	70.7	54.4	37.4	93.4	80.5	65.0
$r_x = r_y$ (in.^{4})	3.15	3.18	3.21	2.58	2.63	2.69

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 46 ksi.

Table 4-4 (continued)
Available Strength in Axial Compression, kips

HSS7–HSS6

Square HSS

F_y = 46 ksi

Shape		HSS7×7×								HSS6×6×			
		5/16		1/4		3/16^c		1/8^c		5/8		1/2	
t_{design}, in.		0.291		0.233		0.174		0.116		0.581		0.465	
Wt/ft		27.5		22.4		17.1		11.6		42.1		35.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	209	314	170	255	124	187	61.6	92.6	323	485	268	403
	6	199	300	162	244	120	181	60.4	90.8	300	451	250	376
	7	196	295	160	240	119	179	60.0	90.1	292	439	244	366
	8	192	289	156	235	118	177	59.5	89.4	283	426	237	356
	9	188	283	153	230	116	174	58.9	88.5	273	411	229	344
	10	183	276	149	225	114	171	58.3	87.6	263	395	221	332
	11	178	268	145	219	111	166	57.6	86.5	252	378	212	318
	12	173	260	141	212	107	161	56.8	85.3	240	361	202	304
	13	168	252	137	206	104	156	55.9	84.0	228	343	193	290
	14	162	243	132	199	101	151	55.0	82.6	216	324	183	275
	15	156	234	127	191	97.0	146	54.0	81.1	203	306	173	260
	16	150	225	122	184	93.3	140	52.9	79.5	191	287	163	244
	17	143	215	117	176	89.5	134	51.7	77.8	178	268	152	229
	18	137	206	112	168	85.6	129	50.5	75.9	166	249	142	214
	19	130	196	107	161	81.7	123	49.2	73.9	154	231	132	199
	20	124	186	102	153	77.8	117	47.8	71.8	142	213	123	184
	21	117	176	96.4	145	73.9	111	46.3	69.5	130	196	113	170
	22	111	167	91.2	137	70.0	105	44.7	67.2	119	179	104	156
	23	105	157	86.1	129	66.2	99.4	43.0	64.6	109	164	95.2	143
	24	98.3	148	81.1	122	62.4	93.7	41.2	61.9	100	150	87.5	131
	25	92.2	139	76.1	114	58.6	88.1	39.3	59.1	92.2	139	80.6	121
	26	86.2	130	71.3	107	55.0	82.7	37.4	56.1	85.2	128	74.5	112
	27	80.3	121	66.6	100	51.4	77.3	35.3	53.0	79.0	119	69.1	104
	28	74.7	112	61.9	93.1	47.9	72.0	33.0	49.7	73.5	110	64.3	96.6
	29	69.6	105	57.7	86.8	44.7	67.1	30.8	46.3	68.5	103	59.9	90.0
	30	65.1	97.8	54.0	81.1	41.7	62.7	28.8	43.3	64.0	96.2	56.0	84.1
	32	57.2	86.0	47.4	71.3	36.7	55.1	25.3	38.0	56.3	84.6	49.2	73.9
	34	50.7	76.2	42.0	63.1	32.5	48.8	22.4	33.7	49.8	74.9	43.6	65.5
	36	45.2	67.9	37.5	56.3	29.0	43.6	20.0	30.0	44.5	66.8	38.9	58.4
	38	40.6	61.0	33.6	50.5	26.0	39.1	17.9	27.0				
	40	36.6	55.0	30.4	45.6	23.5	35.3	16.2	24.3				

Properties

	HSS7×7×5/16	HSS7×7×1/4	HSS7×7×3/16	HSS7×7×1/8	HSS6×6×5/8	HSS6×6×1/2
A_g (in.^{2})	7.59	6.17	4.67	3.16	11.7	9.74
$I_x = I_y$ (in.^{4})	56.1	46.5	36.0	24.8	55.2	48.3
$r_x = r_y$ (in.^{4})	2.72	2.75	2.77	2.80	2.17	2.23

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS6

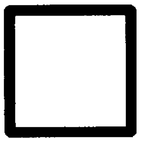
Shape		HSS6×6×									
		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8{}^c$	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Wt/ft		27.4		23.3		19.0		14.5		9.85	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	209	314	177	266	144	217	110	165	59.5	89.4
	6	195	293	166	249	135	203	103	155	57.6	86.6
	7	190	286	162	243	132	199	101	151	57.0	85.6
	8	185	278	158	237	129	193	98.1	147	56.2	84.4
	9	179	270	153	230	125	188	95.2	143	55.3	83.1
	10	173	260	148	222	121	182	92.1	138	54.3	81.6
	11	167	250	142	214	116	175	88.9	134	53.2	80.0
	12	160	240	136	205	112	168	85.4	128	52.0	78.1
	13	152	229	130	196	107	161	81.8	123	50.6	76.1
	14	145	218	124	186	102	153	78.0	117	49.2	73.9
	15	137	206	118	177	96.8	145	74.2	112	47.6	71.6
	16	130	195	111	167	91.6	138	70.4	106	45.9	69.0
	17	122	183	105	157	86.4	130	66.5	99.9	44.1	66.3
	18	114	172	98.3	148	81.2	122	62.5	94.0	42.1	63.3
	19	107	160	91.9	138	76.1	114	58.7	88.2	40.1	60.2
	20	99.2	149	85.7	129	71.0	107	54.8	82.4	37.8	56.8
	21	91.9	138	79.5	120	66.0	99.2	51.1	76.8	35.2	53.0
	22	84.8	128	73.6	111	61.2	91.9	47.4	71.3	32.8	49.3
	23	77.9	117	67.7	102	56.5	84.9	43.9	65.9	30.4	45.7
	24	71.5	107	62.2	93.4	51.9	78.0	40.4	60.7	28.0	42.1
	25	65.9	99.1	57.3	86.1	47.8	71.8	37.2	55.9	25.8	38.8
	26	60.9	91.6	53.0	79.6	44.2	66.4	34.4	51.7	23.9	35.9
	27	56.5	84.9	49.1	73.8	41.0	61.6	31.9	47.9	22.1	33.3
	28	52.5	79.0	45.7	68.7	38.1	57.3	29.7	44.6	20.6	30.9
	29	49.0	73.6	42.6	64.0	35.5	53.4	27.6	41.6	19.2	28.8
	30	45.8	68.8	39.8	59.8	33.2	49.9	25.8	38.8	17.9	26.9
	32	40.2	60.5	35.0	52.6	29.2	43.9	22.7	34.1	15.8	23.7
	34	35.6	53.6	31.0	46.6	25.8	38.8	20.1	30.2	14.0	21.0
	36	31.8	47.8	27.6	41.5	23.1	34.6	17.9	27.0	12.4	18.7
	38	28.5	42.9	24.8	37.3	20.7	31.1	16.1	24.2	11.2	16.8

Properties

A_g (in.^{2})	7.58	6.43	5.24	3.98	2.70
$I_x = I_y$ (in.^{4})	39.5	34.3	28.6	22.3	15.5
$r_x = r_y$ (in.^{4})	2.28	2.31	2.34	2.37	2.39

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 46 ksi.



HSS5½–HSS5

Table 4–4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 46 ksi

Shape		HSS5½×5½×										HSS5×5×	
		3/8		5/16		1/4		3/16		1/8 ^{c}		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
Wt/ft		24.9		21.2		17.3		13.2		9.00		28.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	189	285	161	242	131	197	100	150	58.1	87.4	217	326
	1	189	284	161	241	131	197	99.8	150	58.1	87.3	216	325
	2	188	282	160	240	130	196	99.1	149	57.9	87.0	214	322
	3	186	279	158	237	129	194	98.1	147	57.5	86.5	211	318
	4	183	275	155	234	127	191	96.7	145	57.1	85.8	207	311
	5	179	269	152	229	125	187	94.9	143	56.5	84.9	202	303
	6	175	263	149	224	122	183	92.8	139	55.8	83.8	195	293
	7	170	255	145	217	118	178	90.3	136	54.9	82.5	188	283
	8	164	247	140	210	115	172	87.5	132	53.9	81.0	180	270
	9	158	237	135	203	111	166	84.5	127	52.8	79.3	171	257
	10	151	228	129	195	106	160	81.3	122	51.5	77.4	162	243
	11	144	217	124	186	102	153	77.8	117	50.1	75.3	152	229
	12	137	206	118	177	96.7	145	74.2	111	48.5	72.9	142	214
	13	130	195	111	167	91.7	138	70.4	106	46.8	70.4	132	199
	14	122	183	105	158	86.6	130	66.6	100	45.0	67.6	122	184
	15	114	172	98.5	148	81.4	122	62.7	94.3	42.9	64.5	112	169
	16	107	160	92.1	138	76.2	115	58.8	88.4	40.5	60.8	102	154
	17	99.1	149	85.7	129	71.0	107	54.9	82.6	37.9	56.9	93.0	140
	18	91.6	138	79.4	119	65.9	99.1	51.1	76.8	35.3	53.0	83.8	126
	19	84.3	127	73.2	110	61.0	91.6	47.3	71.1	32.7	49.2	75.2	113
	20	77.2	116	67.2	101	56.1	84.3	43.6	65.6	30.3	45.5	67.9	102
	21	70.3	106	61.4	92.2	51.4	77.3	40.1	60.3	27.9	41.9	61.6	92.6
	22	64.0	96.3	55.9	84.0	46.8	70.4	36.6	55.0	25.5	38.3	56.1	84.3
	23	58.6	88.1	51.2	76.9	42.9	64.4	33.5	50.3	23.3	35.0	51.3	77.2
	24	53.8	80.9	47.0	70.6	39.4	59.2	30.8	46.2	21.4	32.2	47.2	70.9
	25	49.6	74.5	43.3	65.1	36.3	54.5	28.3	42.6	19.7	29.7	43.5	65.3
	26	45.9	68.9	40.0	60.2	33.5	50.4	26.2	39.4	18.2	27.4	40.2	60.4
	27	42.5	63.9	37.1	55.8	31.1	46.7	24.3	36.5	16.9	25.4	37.3	56.0
	28	39.5	59.4	34.5	51.9	28.9	43.5	22.6	34.0	15.7	23.6	34.6	52.1
	29	36.9	55.4	32.2	48.4	27.0	40.5	21.1	31.7	14.7	22.0	32.3	48.5
	30	34.4	51.8	30.1	45.2	25.2	37.9	19.7	29.6	13.7	20.6	30.2	45.4

Properties

A_g (in.^{2})	6.88	5.85	4.77	3.63	2.46	7.88
$I_x = I_y$ (in.^{4})	29.7	25.9	21.7	17.0	11.8	26.0
$r_x = r_y$ (in.^{4})	2.08	2.11	2.13	2.16	2.19	1.82

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS5–HSS4½

Shape		HSS5×5×										HSS4½×4½×	
		3/8		5/16		1/4		3/16		1/8^c		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
Wt/ft		22.3		19.0		15.6		12.0		8.15		24.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	170	256	145	218	119	178	90.4	136	56.5	84.9	191	288
	1	170	255	145	217	118	178	90.1	135	56.4	84.8	191	287
	2	168	253	143	216	117	176	89.5	134	56.1	84.4	189	283
	3	166	250	142	213	116	174	88.3	133	55.7	83.7	185	278
	4	163	245	139	209	114	171	86.8	130	55.1	82.8	180	271
	5	159	239	136	204	111	167	84.8	128	54.3	81.7	174	262
	6	154	232	132	198	108	162	82.5	124	53.4	80.3	167	252
	7	149	223	127	191	104	157	79.9	120	52.3	78.6	159	240
	8	143	214	122	184	100	151	76.9	116	51.0	76.6	151	227
	9	136	205	117	175	96.0	144	73.7	111	49.5	74.4	142	213
	10	129	194	111	167	91.4	137	70.2	106	47.8	71.9	132	198
	11	122	183	105	158	86.5	130	66.6	100	45.7	68.6	122	183
	12	114	172	98.6	148	81.5	122	62.8	94.4	43.2	64.9	112	168
	13	107	160	92.2	139	76.4	115	59.0	88.6	40.6	61.0	102	153
	14	99.1	149	85.8	129	71.2	107	55.1	82.8	38.0	57.1	92.3	139
	15	91.5	138	79.4	119	66.0	99.2	51.2	76.9	35.4	53.2	82.8	124
	16	84.0	126	73.0	110	60.9	91.5	47.3	71.1	32.8	49.3	73.7	111
	17	76.7	115	66.9	100	55.9	84.0	43.6	65.5	30.2	45.4	65.3	98.1
	18	69.6	105	60.9	91.5	51.0	76.7	39.9	59.9	27.7	41.7	58.2	87.5
	19	62.7	94.3	55.0	82.7	46.4	69.7	36.3	54.6	25.3	38.1	52.3	78.6
	20	56.6	85.1	49.7	74.7	41.8	62.9	32.8	49.3	23.0	34.5	47.2	70.9
	21	51.4	77.2	45.1	67.7	37.9	57.0	29.8	44.8	20.8	31.3	42.8	64.3
	22	46.8	70.3	41.1	61.7	34.6	51.9	27.1	40.8	19.0	28.5	39.0	58.6
	23	42.8	64.3	37.6	56.5	31.6	47.5	24.8	37.3	17.4	26.1	35.7	53.6
	24	39.3	59.1	34.5	51.9	29.0	43.7	22.8	34.3	16.0	24.0	32.8	49.2
	25	36.2	54.5	31.8	47.8	26.8	40.2	21.0	31.6	14.7	22.1	30.2	45.4
	26	33.5	50.4	29.4	44.2	24.7	37.2	19.4	29.2	13.6	20.4	27.9	42.0
	27	31.1	46.7	27.3	41.0	22.9	34.5	18.0	27.1	12.6	18.9		
	28	28.9	43.4	25.3	38.1	21.3	32.1	16.8	25.2	11.7	17.6		
	29	26.9	40.5	23.6	35.5	19.9	29.9	15.6	23.5	10.9	16.4		
	40	25.2	37.8	22.1	33.2	18.6	27.9	14.6	21.9	10.2	15.3		

Properties

	3/8	5/16	1/4	3/16	1/8^c	1/2
A_g (in.^{2})	6.18	5.26	4.30	3.28	2.23	6.95
$I_x = I_y$ (in.^{4})	21.7	19.0	16.0	12.6	8.80	18.1
$r_x = r_y$ (in.^{4})	1.87	1.90	1.93	1.96	1.99	1.61

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

Table 4–4 (continued)

Available Strength in Axial Compression, kips

F_y = 46 ksi

HSS4½–HSS4

Square HSS

Shape		HSS4½×4½×										HSS4×4×	
		3/8		5/16		1/4		3/16		1/8^c		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
Wt/ft		19.7		16.9		13.9		10.7		7.30		21.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	151	227	129	194	106	159	80.8	121	54.4	81.8	166	249
	1	150	226	129	193	105	158	80.5	121	54.3	81.6	165	248
	2	149	224	127	191	104	157	79.8	120	54.0	81.1	163	244
	3	146	220	125	188	103	154	78.5	118	53.4	80.3	159	238
	4	143	215	122	184	100	151	76.8	115	52.5	78.8	153	230
	5	138	208	119	178	97.4	146	74.7	112	51.0	76.7	147	221
	6	133	200	114	172	94.0	141	72.1	108	49.3	74.2	139	209
	7	127	191	109	164	90.1	135	69.2	104	47.4	71.3	131	196
	8	121	182	104	156	85.8	129	66.0	99.3	45.3	68.1	121	182
	9	114	171	98.2	148	81.2	122	62.6	94.1	43.0	64.7	112	168
	10	107	160	92.1	138	76.3	115	59.0	88.6	40.6	61.0	102	153
	11	99.1	149	85.9	129	71.3	107	55.2	83.0	38.1	57.2	91.8	138
	12	91.5	138	79.5	119	66.2	99.4	51.3	77.2	35.5	53.3	82.1	123
	13	83.9	126	73.1	110	61.0	91.7	47.5	71.3	32.9	49.4	72.6	109
	14	76.4	115	66.7	100	55.9	84.0	43.6	65.5	30.3	45.5	63.6	95.6
	15	69.1	104	60.5	91.0	50.8	76.4	39.8	59.8	27.7	41.7	55.4	83.3
	16	62.0	93.2	54.5	82.0	46.0	69.1	36.1	54.2	25.2	37.9	48.7	73.2
	17	55.2	82.9	48.7	73.2	41.3	62.0	32.5	48.9	22.8	34.3	43.1	64.8
	18	49.2	74.0	43.5	65.3	36.8	55.3	29.1	43.7	20.5	30.7	38.5	57.8
	19	44.2	66.4	39.0	58.6	33.0	49.7	26.1	39.2	18.4	27.6	34.5	51.9
	20	39.9	59.9	35.2	52.9	29.8	44.8	23.5	35.4	16.6	24.9	31.2	46.8
	21	36.2	54.3	31.9	48.0	27.0	40.6	21.4	32.1	15.0	22.6	28.3	42.5
	22	32.9	49.5	29.1	43.7	24.6	37.0	19.5	29.3	13.7	20.6	25.8	38.7
	23	30.1	45.3	26.6	40.0	22.5	33.9	17.8	26.8	12.5	18.8	23.6	35.4
	24	27.7	41.6	24.4	36.7	20.7	31.1	16.4	24.6	11.5	17.3		
	25	25.5	38.3	22.5	33.9	19.1	28.7	15.1	22.7	10.6	15.9		
	26	23.6	35.5	20.8	31.3	17.6	26.5	13.9	20.9	9.80	14.7		
	27	21.9	32.9	19.3	29.0	16.4	24.6	12.9	19.4	9.09	13.7		
	28			18.0	27.0	15.2	22.9	12.0	18.1	8.45	12.7		
	29							11.2	16.8	7.88	11.8		

Properties

A_g (in.^{2})	5.48	4.68	3.84	2.93	2.00	6.02
$I_x = I_y$ (in.^{4})	15.3	13.5	11.4	9.02	6.35	11.9
$r_x = r_y$ (in.^{4})	1.67	1.70	1.73	1.75	1.78	1.41

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 46 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS4

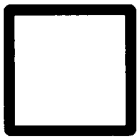
Shape		HSS4×4×									
		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Wt/ft		17.2		14.8		12.2		9.40		6.45	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	132	198	113	170	92.9	140	71.2	107	48.7	73.2
	1	131	197	112	169	92.5	139	70.9	107	48.5	72.9
	2	129	194	111	167	91.3	137	70.1	105	47.9	72.0
	3	127	190	109	163	89.4	134	68.7	103	47.0	70.7
	4	123	184	105	158	86.8	131	66.8	100	45.7	68.8
	5	118	177	101	152	83.6	126	64.4	96.7	44.2	66.4
	6	112	168	96.6	145	79.9	120	61.6	92.6	42.3	63.6
	7	106	159	91.3	137	75.6	114	58.4	87.8	40.2	60.5
	8	98.7	148	85.5	129	71.0	107	55.0	82.7	38.0	57.0
	9	91.4	137	79.4	119	66.2	99.4	51.4	77.2	35.5	53.4
	10	83.9	126	73.1	110	61.1	91.8	47.6	71.5	33.0	49.6
	11	76.3	115	66.8	100	56.0	84.1	43.7	65.7	30.4	45.7
	12	68.8	103	60.4	90.8	50.8	76.4	39.8	59.9	27.8	41.8
	13	61.5	92.4	54.2	81.5	45.8	68.8	36.0	54.1	25.2	37.9
	14	54.4	81.8	48.2	72.4	40.9	61.5	32.3	48.6	22.7	34.1
	15	47.6	71.6	42.4	63.7	36.2	54.4	28.7	43.2	20.3	30.5
	16	41.9	62.9	37.3	56.0	31.8	47.8	25.3	38.1	17.9	27.0
	17	37.1	55.7	33.0	49.6	28.2	42.4	22.4	33.7	15.9	23.9
	18	33.1	49.7	29.4	44.3	25.1	37.8	20.0	30.1	14.2	21.3
	19	29.7	44.6	26.4	39.7	22.6	33.9	18.0	27.0	12.7	19.1
	20	26.8	40.3	23.9	35.9	20.4	30.6	16.2	24.4	11.5	17.3
	21	24.3	36.5	21.6	32.5	18.5	27.8	14.7	22.1	10.4	15.7
	22	22.1	33.3	19.7	29.6	16.8	25.3	13.4	20.1	9.49	14.3
	23	20.3	30.5	18.0	27.1	15.4	23.1	12.3	18.4	8.68	13.0
	24	18.6	28.0	16.6	24.9	14.1	21.3	11.3	16.9	7.97	12.0
	25					13.0	19.6	10.4	15.6	7.35	11.0
	26									6.79	10.2

Properties

	3/8	5/16	1/4	3/16	1/8
A_g (in.^{2})	4.78	4.10	3.37	2.58	1.77
$I_x = I_y$ (in.^{4})	10.3	9.14	7.80	6.21	4.40
$r_x = r_y$ (in.^{4})	1.47	1.49	1.52	1.55	1.58

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



HSS3½

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 46 ksi

Shape		**HSS3½×3½×**									
		3/8		**5/16**		**1/4**		**3/16**		**1/8**	
t_{design}, in.		**0.349**		**0.291**		**0.233**		**0.174**		**0.116**	
Wt/ft		**14.6**		**12.7**		**10.5**		**8.13**		**5.60**	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y	**0**	**113**	169	**96.9**	146	**80.0**	120	**61.6**	92.6	**42.3**	63.6
	1	**112**	168	**96.3**	145	**79.6**	120	**61.3**	92.1	**42.1**	63.2
	2	**110**	165	**94.6**	142	**78.2**	118	**60.3**	90.6	**41.4**	62.3
	3	**107**	160	**91.9**	138	**76.1**	114	**58.7**	88.3	**40.4**	60.7
	4	**102**	153	**88.2**	133	**73.2**	110	**56.6**	85.0	**39.0**	58.6
	5	**96.6**	145	**83.7**	126	**69.6**	105	**53.9**	81.0	**37.2**	55.9
	6	**90.4**	136	**78.5**	118	**65.4**	98.4	**50.8**	76.4	**35.2**	52.8
	7	**83.5**	125	**72.8**	109	**60.9**	91.5	**47.4**	71.3	**32.9**	49.4
	8	**76.2**	115	**66.7**	100	**56.0**	84.1	**43.8**	65.8	**30.5**	45.8
	9	**68.7**	103	**60.4**	90.8	**50.9**	76.5	**39.9**	60.0	**27.9**	41.9
	10	**61.2**	91.9	**54.1**	81.3	**45.8**	68.8	**36.1**	54.2	**25.3**	38.0
	11	**53.8**	80.9	**47.8**	71.9	**40.7**	61.2	**32.3**	48.5	**22.7**	34.2
	12	**46.8**	70.3	**41.8**	62.9	**35.8**	53.8	**28.5**	42.9	**20.2**	30.4
	13	**40.1**	60.3	**36.1**	54.2	**31.1**	46.8	**25.0**	37.5	**17.8**	26.7
	14	**34.6**	52.0	**31.1**	46.8	**26.8**	40.3	**21.6**	32.4	**15.4**	23.2
	15	**30.1**	45.3	**27.1**	40.7	**23.4**	35.1	**18.8**	28.2	**13.4**	20.2
	16	**26.5**	39.8	**23.8**	35.8	**20.5**	30.9	**16.5**	24.8	**11.8**	17.8
	17	**23.5**	35.2	**21.1**	31.7	**18.2**	27.4	**14.6**	22.0	**10.5**	15.7
	18	**20.9**	31.4	**18.8**	28.3	**16.2**	24.4	**13.0**	19.6	**9.33**	14.0
	19	**18.8**	28.2	**16.9**	25.4	**14.6**	21.9	**11.7**	17.6	**8.38**	12.6
	20	**16.9**	25.5	**15.2**	22.9	**13.2**	19.8	**10.6**	15.9	**7.56**	11.4
	21	**15.4**	23.1	**13.8**	20.8	**11.9**	17.9	**9.59**	14.4	**6.86**	10.3
	22					**10.9**	16.3	**8.74**	13.1	**6.25**	9.39

Properties

	3/8	5/16	1/4	3/16	1/8
A_g (in.^{2})	4.09	3.52	2.91	2.24	1.54
$I_x = I_y$ (in.^{4})	6.49	5.84	5.04	4.05	2.90
$r_x = r_y$ (in.^{4})	1.26	1.29	1.32	1.35	1.37

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi

Table 4-4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS3

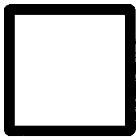
Shape		HSS3×3×									
		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Wt/ft		12.1		10.5		8.78		6.85		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	93.3	140	80.8	122	67.2	101	52.0	78.2	35.9	54.0
	1	92.5	139	80.2	121	66.7	100	51.6	77.6	35.7	53.6
	2	90.1	135	78.2	118	65.1	97.9	50.5	75.9	34.9	52.5
	3	86.3	130	75.1	113	62.6	94.1	48.7	73.1	33.7	50.6
	4	81.2	122	70.9	107	59.3	89.1	46.2	69.4	32.1	48.2
	5	75.1	113	65.8	98.9	55.2	83.0	43.2	64.9	30.1	45.2
	6	68.3	103	60.1	90.3	50.7	76.2	39.8	59.8	27.8	41.8
	7	61.0	91.6	54.0	81.2	45.8	68.8	36.1	54.3	25.4	38.1
	8	53.5	80.5	47.7	71.7	40.7	61.2	32.3	48.6	22.8	34.3
	9	46.2	69.4	41.5	62.4	35.7	53.6	28.5	42.8	20.2	30.4
	10	39.2	58.9	35.5	53.3	30.7	46.2	24.7	37.2	17.7	26.6
	11	32.6	49.0	29.8	44.8	26.1	39.2	21.2	31.8	15.2	22.9
	12	27.4	41.2	25.0	37.6	21.9	32.9	17.8	26.8	12.9	19.4
	13	23.3	35.1	21.3	32.1	18.7	28.0	15.2	22.8	11.0	16.5
	14	20.1	30.2	18.4	27.6	16.1	24.2	13.1	19.7	9.49	14.3
	15	17.5	26.4	16.0	24.1	14.0	21.1	11.4	17.2	8.27	12.4
	16	15.4	23.2	14.1	21.2	12.3	18.5	10.00	15.1	7.27	10.9
	17	13.6	20.5	12.5	18.7	10.9	16.4	8.89	13.4	6.44	9.68
	18			11.1	16.7	9.73	14.6	7.93	11.9	5.74	8.63
	19							7.12	10.7	5.15	7.75

Properties

A_g (in.^{2})	3.39	2.94	2.44	1.89	1.30
$I_x = I_y$ (in.^{4})	3.78	3.45	3.02	2.46	1.78
$r_x = r_y$ (in.^{4})	1.06	1.08	1.11	1.14	1.17

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



HSS2½–HSS2¼

Table 4–4 (continued)
Available Strength in Axial Compression, kips
Square HSS

F_y = 46 ksi

Shape		HSS2½×2½×								HSS2¼×2¼×	
		5/16		1/4		3/16		1/8		1/4	
t_{design}, in.		0.291		0.233		0.174		0.116		0.233	
Wt/ft		8.40		7.08		5.57		3.90		6.23	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	64.8	97.4	54.3	81.7	42.4	63.8	29.5	44.4	47.9	72.0
	1	64.0	96.2	53.7	80.7	42.0	63.1	29.2	43.9	47.2	71.0
	2	61.7	92.7	51.9	77.9	40.6	61.0	28.3	42.6	45.2	67.9
	3	57.9	87.0	48.9	73.5	38.4	57.8	26.9	40.4	41.9	63.0
	4	53.1	79.7	45.0	67.7	35.6	53.5	25.0	37.6	37.8	56.7
	5	47.4	71.3	40.5	60.9	32.2	48.4	22.8	34.2	33.0	49.6
	6	41.3	62.1	35.6	53.5	28.5	42.9	20.3	30.5	28.0	42.1
	7	35.1	52.8	30.6	46.0	24.7	37.2	17.7	26.7	23.1	34.7
	8	29.1	43.8	25.6	38.5	21.0	31.5	15.2	22.8	18.4	27.7
	9	23.5	35.3	21.0	31.5	17.4	26.1	12.7	19.1	14.6	21.9
	10	19.0	28.6	17.0	25.5	14.1	21.2	10.4	15.7	11.8	17.7
	11	15.7	23.6	14.0	21.1	11.7	17.5	8.61	12.9	9.76	14.7
	12	13.2	19.9	11.8	17.7	9.81	14.7	7.24	10.9	8.20	12.3
	13	11.3	16.9	10.1	15.1	8.36	12.6	6.17	9.27	6.99	10.5
	14	9.71	14.6	8.67	13.0	7.21	10.8	5.32	7.99		
	15			7.55	11.4	6.28	9.44	4.63	6.96		
	16							4.07	6.12		

Properties

	5/16	1/4	3/16	1/8	HSS2¼×2¼×1/4
A_g (in.^{2})	2.35	1.97	1.54	1.07	1.74
$I_x = I_y$ (in.^{4})	1.82	1.63	1.35	0.998	1.13
$r_x = r_y$ (in.^{4})	0.880	0.908	0.937	0.965	0.806

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi

Table 4–4 (continued)
Available Strength in Axial Compression, kips
Square HSS

HSS2¼–HSS2

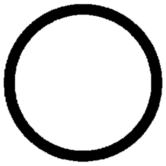
Shape	HSS2¼×2¼×				HSS2×2×					
	3/16		1/8		1/4		3/16		1/8	
t_{design}, in.	0.174		0.116		0.233		0.174		0.116	
Wt/ft	4.94		3.47		5.38		4.30		3.04	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y										
0	37.7	56.6	26.3	39.6	41.5	62.4	32.9	49.4	23.1	34.8
1	37.1	55.8	26.0	39.0	40.7	61.2	32.3	48.5	22.7	34.2
2	35.6	53.5	25.0	37.6	38.4	57.7	30.6	45.9	21.6	32.5
3	33.2	49.9	23.4	35.2	34.8	52.3	27.9	42.0	19.9	29.9
4	30.1	45.3	21.4	32.1	30.4	45.6	24.6	37.0	17.7	26.6
5	26.6	40.0	19.0	28.6	25.5	38.3	20.9	31.5	15.2	22.9
6	22.8	34.3	16.5	24.8	20.5	30.9	17.2	25.8	12.7	19.0
7	19.1	28.6	13.9	20.9	15.9	23.9	13.6	20.4	10.2	15.3
8	15.5	23.3	11.5	17.2	12.2	18.3	10.4	15.7	7.93	11.9
9	12.3	18.5	9.17	13.8	9.63	14.5	8.26	12.4	6.26	9.42
10	9.95	15.0	7.43	11.2	7.80	11.7	6.69	10.1	5.07	7.63
11	8.22	12.4	6.14	9.23	6.44	9.68	5.53	8.31	4.19	6.30
12	6.91	10.4	5.16	7.75			4.64	6.98	3.52	5.30
13	5.89	8.85	4.40	6.61						
14			3.79	5.70						

Properties

A_g (in.^{2})	1.37	0.956	1.51	1.19	0.840
$I_x = I_y$ (in.^{4})	0.953	0.712	0.747	0.641	0.486
$r_x = r_y$ (in.^{4})	0.835	0.863	0.704	0.733	0.761

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



HSS18.000–
HSS16.000

Table 4–5
Available Strength in Axial Compression, kips
Round HSS

F_y = 42 ksi

Shape	HSS18.000×				HSS16.000×							
	0.500		0.375		0.625		0.500		0.438		0.375	
t_{design}, in.	0.465		0.349		0.581		0.465		0.407		0.349	
Wt/ft	93.5		70.7		103		82.8		72.9		62.6	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y												
0	644	968	487	732	708	1060	571	858	501	754	432	649
6	639	960	483	726	700	1050	565	849	496	746	427	642
7	637	957	481	723	698	1050	563	846	494	743	426	640
8	635	954	480	721	694	1040	560	842	492	740	424	637
9	632	950	478	718	691	1040	557	838	490	736	422	634
10	630	946	476	715	687	1030	554	833	487	732	419	630
11	627	942	474	712	683	1030	551	828	484	728	417	626
12	623	937	471	708	678	1020	547	822	481	723	414	622
13	620	931	468	704	673	1010	543	816	477	718	411	618
14	616	926	466	700	668	1000	539	810	474	712	408	613
15	612	919	462	695	662	995	534	803	470	706	404	608
16	607	913	459	690	656	986	530	796	465	700	401	602
17	603	906	456	685	650	976	524	788	461	693	397	597
18	598	899	452	680	643	966	519	780	456	686	393	591
19	593	891	448	674	636	956	513	772	451	679	389	584
20	588	883	444	668	628	945	508	763	446	671	385	578
21	582	875	440	662	621	933	502	754	441	663	380	571
22	576	866	436	655	613	921	495	744	436	655	375	564
23	570	857	432	649	605	909	489	735	430	646	370	557
24	564	848	427	642	596	896	482	725	424	637	365	549
25	558	839	422	635	588	884	475	714	418	628	360	542
26	551	829	417	627	579	870	468	704	412	619	355	534
27	545	819	412	620	570	857	461	693	406	610	350	526
28	538	809	407	612	561	843	454	682	399	600	344	517
29	531	798	402	604	551	829	446	671	393	590	339	509
30	524	787	397	596	542	814	438	659	386	580	333	500
32	509	765	386	580	522	785	423	636	372	560	321	483
34	494	742	374	563	502	755	407	611	358	538	309	465
36	478	719	363	545	482	724	390	587	344	517	297	446
38	462	695	351	527	461	693	374	562	329	495	284	428
40	446	670	338	509	440	661	357	537	315	473	272	409

Properties

A_g (in.^{2})	25.6	19.4	28.1	22.7	19.9	17.2
I (in.^{4})	985	754	838	685	606	526
r (in.)	6.20	6.24	5.46	5.49	5.51	5.53

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 42 ksi

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS16.000–
HSS14.000

Shape		HSS16.000×				HSS14.000×							
		0.312		0.250		0.625		0.500		0.375		0.312	
t_{design}, in.		0.291		0.233		0.581		0.465		0.349		0.291	
Wt/ft		52.3		42.1		89.4		72.2		54.6		45.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	361	543	290	436	616	926	497	747	376	566	315	474
	6	357	537	287	432	607	913	490	737	371	558	311	467
	7	356	535	286	430	604	908	488	733	369	555	309	465
	8	355	533	285	428	601	903	485	729	367	552	308	462
	9	353	530	284	426	597	897	482	724	365	549	306	460
	10	351	528	282	424	592	890	478	719	362	545	304	456
	11	349	524	280	421	587	883	475	713	360	540	301	453
	12	347	521	279	419	582	875	470	707	356	536	299	449
	13	344	517	277	416	576	866	466	700	353	531	296	445
	14	341	513	275	413	570	857	461	693	349	525	293	440
	15	339	509	272	409	564	848	456	685	346	519	290	435
	16	336	504	270	406	557	837	451	677	342	513	286	430
	17	332	500	267	402	550	827	445	669	337	507	283	425
	18	329	495	265	398	542	815	439	660	333	500	279	419
	19	326	489	262	394	535	804	433	650	328	493	275	414
	20	322	484	259	389	527	791	426	641	323	486	271	408
	21	318	478	256	385	518	779	419	630	318	479	267	401
	22	314	473	253	380	509	766	413	620	313	471	263	395
	23	310	466	250	375	501	752	405	609	308	463	258	388
	24	306	460	246	370	491	739	398	598	303	455	254	381
	25	302	454	243	365	482	725	391	587	297	446	249	374
	26	298	447	239	360	473	710	383	576	291	438	244	367
	27	293	440	236	355	463	696	375	564	285	429	240	360
	28	288	434	232	349	453	681	367	552	280	420	235	353
	29	284	427	228	343	443	666	359	540	274	411	230	345
	30	279	419	225	338	433	650	351	528	268	402	225	338
	32	269	405	217	326	412	620	335	503	255	384	214	322
	34	259	390	209	314	391	588	318	478	243	365	204	307
	36	249	374	201	302	371	557	302	453	230	346	194	291
	38	239	359	192	289	350	526	285	428	218	327	183	275
	40	228	343	184	277	329	494	268	403	205	308	173	259

Properties

A_g (in.^{2})	14.4	11.5	24.5	19.8	15.0	12.5
I (in.^{4})	443	359	552	453	349	295
r (in.)	5.55	5.58	4.75	4.79	4.83	4.85

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

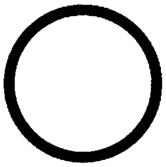


Table 4–5 (continued)
Available Strength in Axial Compression, kips

Round HSS

HSS14.000–
HSS10.750

F_y = 42 ksi

Shape		HSS14.000×		HSS12.750×						HSS10.750×			
		0.250		0.500		0.375		0.250		0.500		0.375	
t_{design}, in.		0.233		0.465		0.349		0.233		0.465		0.349	
Wt/ft		36.7		65.5		49.6		33.4		54.8		41.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	253	381	451	678	342	514	230	346	378	568	287	431
	6	250	376	444	667	336	506	227	341	369	554	280	421
	7	249	374	441	663	334	503	225	339	366	550	278	417
	8	247	372	438	658	332	499	224	336	362	544	275	413
	9	246	370	435	653	329	495	222	334	358	538	272	409
	10	244	367	431	647	327	491	220	331	353	531	269	404
	11	242	364	426	641	323	486	218	328	349	524	265	398
	12	240	361	422	634	320	481	216	325	343	516	261	392
	13	238	358	417	627	316	476	214	321	338	507	257	386
	14	236	354	412	619	312	470	211	317	332	498	252	379
	15	233	350	406	611	308	463	208	313	325	489	248	372
	16	230	346	400	602	304	457	205	309	319	479	243	365
	17	228	342	394	593	299	450	202	304	312	468	237	357
	18	225	338	388	583	295	443	199	299	304	457	232	349
	19	221	333	381	573	290	435	196	294	297	446	227	341
	20	218	328	374	563	285	428	192	289	289	435	221	332
	21	215	323	367	552	279	420	189	284	282	423	215	323
	22	212	318	360	541	274	411	185	278	274	411	209	314
	23	208	313	352	530	268	403	181	273	265	399	203	305
	24	204	307	345	518	262	394	178	267	257	387	197	296
	25	201	302	337	506	257	386	174	261	249	374	191	287
	26	197	296	329	494	251	377	170	255	241	362	184	277
	27	193	290	321	482	245	368	166	249	232	349	178	268
	28	189	284	313	470	238	358	162	243	224	337	172	258
	29	185	278	304	458	232	349	158	237	216	324	166	249
	30	181	272	296	445	226	340	153	231	207	311	159	239
	32	173	260	279	420	214	321	145	218	191	287	147	221
	34	165	247	263	395	201	302	137	206	175	263	135	203
	36	156	235	246	370	188	283	128	193	159	239	123	185
	38	148	222	230	345	176	265	120	180	144	216	112	168
	40	139	210	213	321	164	246	112	168	130	195	101	151

Properties

	HSS14.000×0.250	HSS12.750×0.500	HSS12.750×0.375	HSS12.750×0.250	HSS10.750×0.500	HSS10.750×0.375
A_g (in.^{2})	10.1	17.9	13.6	9.16	15.0	11.4
I (in.^{4})	239	339	262	180	199	154
r (in.)	4.87	4.35	4.39	4.43	3.64	3.68

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 42 ksi

Table 4-5 (continued)
Available Strength in Axial Compression, kips
Round HSS

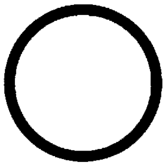
HSS10.750–HSS10.000

Shape		HSS10.750×		HSS10.000×									
		0.250		0.625		0.500		0.375		0.312		0.250	
t_{design}, in.		0.233		0.581		0.465		0.349		0.291		0.233	
Wt/ft		28.1		62.6		50.8		38.6		32.3		26.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	194	291	432	650	350	527	266	400	223	336	180	270
	6	189	284	420	632	341	512	259	389	217	327	175	263
	7	188	282	416	625	337	507	256	385	215	323	173	261
	8	186	279	411	618	333	501	254	381	213	320	171	258
	9	184	276	405	609	329	494	250	376	210	316	169	254
	10	182	273	399	600	324	487	247	371	207	311	167	251
	11	179	269	393	590	319	479	243	365	204	306	164	247
	12	177	265	386	580	313	471	239	359	200	301	162	243
	13	174	261	378	568	307	462	234	352	197	296	159	238
	14	171	257	370	556	301	452	229	345	193	290	155	234
	15	168	252	362	543	294	442	224	337	189	283	152	229
	16	164	247	353	530	287	432	219	329	184	277	149	224
	17	161	242	344	517	280	421	214	321	180	270	145	218
	18	157	237	334	502	272	409	208	313	175	263	141	213
	19	154	231	325	488	265	398	202	304	170	256	138	207
	20	150	225	315	473	257	386	196	295	165	249	134	201
	21	146	219	305	458	249	374	190	286	160	241	130	195
	22	142	214	294	442	241	362	184	277	155	233	126	189
	23	138	207	284	427	232	349	178	268	150	226	121	183
	24	134	201	274	411	224	337	172	258	145	218	117	176
	25	130	195	263	396	216	324	166	249	140	210	113	170
	26	126	189	253	380	207	312	159	240	134	202	109	164
	27	121	183	242	364	199	299	153	230	129	194	105	157
	28	117	176	232	349	191	286	147	221	124	186	101	151
	29	113	170	222	333	182	274	141	211	119	179	96.4	145
	30	109	164	212	318	174	262	134	202	114	171	92.3	139
	32	101	151	192	288	158	238	122	184	104	156	84.2	127
	34	92.5	139	173	259	143	215	111	166	93.8	141	76.3	115
	36	84.5	127	154	232	128	192	99.4	149	84.3	127	68.7	103
	38	76.9	116	138	208	115	172	89.2	134	75.7	114	61.7	92.7
	40	69.5	104	125	188	104	156	80.5	121	68.3	103	55.6	83.6

Properties

	HSS10.750×0.250	HSS10.000×0.625	HSS10.000×0.500	HSS10.000×0.375	HSS10.000×0.312	HSS10.000×0.250
A_g (in.^{2})	7.70	17.2	13.9	10.6	8.88	7.15
I (in.^{4})	106	191	159	123	105	85.3
r (in.)	3.72	3.34	3.38	3.41	3.43	3.45

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$



HSS10.000–
HSS9.625

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

F_y = 42 ksi

Shape	HSS10.000×		HSS9.625×									
	0.188		0.500		0.375		0.312		0.250		0.188	
t_{design}, in.	0.174		0.465		0.349		0.291		0.233		0.174	
Wt/ft	19.7		48.8		37.1		31.1		25.1		19.0	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y												
0	135	203	337	506	256	384	215	323	173	260	130	195
6	132	198	326	491	248	373	208	313	168	252	126	190
7	130	196	323	485	246	369	206	310	166	250	125	188
8	129	194	319	479	243	365	204	306	164	247	124	186
9	127	191	314	473	239	360	201	302	162	244	122	183
10	126	189	309	465	236	354	198	297	160	240	120	180
11	124	186	304	457	232	348	195	292	157	236	118	177
12	122	183	298	448	227	342	191	287	154	232	116	174
13	119	179	292	439	223	335	187	281	151	227	114	171
14	117	176	285	429	218	327	183	275	148	222	111	167
15	115	172	279	419	213	320	179	269	144	217	109	163
16	112	168	271	408	207	312	174	262	141	212	106	159
17	109	164	264	397	202	303	170	255	137	206	103	155
18	107	160	256	385	196	295	165	248	133	200	101	151
19	104	156	248	373	190	286	160	241	129	195	97.6	147
20	101	151	240	361	184	277	155	233	125	189	94.7	142
21	97.8	147	232	349	178	268	150	226	121	182	91.6	138
22	94.8	142	224	337	172	258	145	218	117	176	88.6	133
23	91.7	138	216	324	166	249	140	210	113	170	85.5	128
24	88.6	133	207	312	159	240	134	202	109	164	82.3	124
25	85.5	128	199	299	153	230	129	194	105	157	79.2	119
26	82.3	124	191	286	147	221	124	186	101	151	76.1	114
27	79.2	119	182	274	141	211	119	179	96.4	145	72.9	110
28	76.1	114	174	262	134	202	114	171	92.2	139	69.8	105
29	73.0	110	166	249	128	193	108	163	88.1	132	66.8	100
30	69.9	105	158	237	122	184	103	155	84.0	126	63.7	95.8
32	63.8	95.9	142	214	110	166	93.5	141	76.1	114	57.8	86.8
34	57.9	87.1	127	191	98.9	149	84.0	126	68.4	103	52.0	78.2
36	52.2	78.5	113	170	88.2	133	74.9	113	61.1	91.8	46.5	69.8
38	46.9	70.5	102	153	79.2	119	67.2	101	54.8	82.4	41.7	62.7
40	42.3	63.6	91.8	138	71.5	107	60.7	91.2	49.5	74.4	37.6	56.6

Properties

	HSS10.000×0.188	HSS9.625×0.500	HSS9.625×0.375	HSS9.625×0.312	HSS9.625×0.250	HSS9.625×0.188
A_g (in.^{2})	5.37	13.4	10.2	8.53	6.87	5.17
I (in.^{4})	64.8	141	110	93.0	75.9	57.7
r (in.)	3.47	3.24	3.28	3.30	3.32	3.34

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 42 ksi

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS8.625

Shape		HSS8.625×											
		0.625		0.500		0.375		0.322		0.250		0.188	
t_{design}, in.		0.581		0.465		0.349		0.300		0.233		0.174	
Wt/ft		53.5		43.4		33.1		28.6		22.4		17.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	369	555	300	451	228	343	197	297	154	232	116	175
	6	355	534	289	434	220	330	190	286	149	224	112	169
	7	350	526	285	428	217	326	188	282	147	221	111	166
	8	344	518	280	421	214	321	185	278	145	218	109	164
	9	338	508	275	414	210	316	182	273	142	214	107	161
	10	331	498	270	405	206	309	178	268	140	210	105	158
	11	324	487	264	396	201	303	174	262	137	206	103	155
	12	316	475	257	387	197	296	170	256	134	201	101	151
	13	307	462	251	377	192	288	166	250	130	196	98.3	148
	14	298	448	244	366	186	280	162	243	127	191	95.7	144
	15	289	435	236	355	181	272	157	236	123	185	93.0	140
	16	280	420	229	344	175	263	152	228	119	180	90.2	136
	17	270	405	221	332	169	255	147	221	116	174	87.3	131
	18	260	390	213	320	163	246	142	213	112	168	84.3	127
	19	249	375	205	307	157	236	137	205	108	162	81.3	122
	20	239	359	196	295	151	227	131	197	103	155	78.2	118
	21	229	344	188	282	145	218	126	189	99.2	149	75.1	113
	22	218	328	180	270	139	208	120	181	95.0	143	71.9	108
	23	208	312	171	257	132	199	115	173	90.8	137	68.8	103
	24	197	297	163	245	126	189	110	165	86.7	130	65.7	98.7
	25	187	281	155	232	120	180	104	157	82.5	124	62.6	94.0
	26	177	266	147	220	114	171	99.1	149	78.4	118	59.5	89.4
	27	167	251	139	208	108	162	93.8	141	74.3	112	56.4	84.8
	28	157	237	131	196	102	153	88.7	133	70.3	106	53.5	80.3
	29	148	222	123	185	95.9	144	83.7	126	66.4	99.8	50.5	75.9
	30	138	208	115	174	90.2	136	78.8	118	62.6	94.1	47.7	71.6
	32	122	183	101	152	79.3	119	69.4	104	55.2	82.9	42.1	63.2
	34	108	162	89.9	135	70.3	106	61.5	92.4	48.9	73.4	37.3	56.0
	36	96.1	145	80.2	120	62.7	94.2	54.8	82.4	43.6	65.5	33.2	49.9
	38	86.3	130	72.0	108	56.3	84.6	49.2	73.9	39.1	58.8	29.8	44.8
	40	77.9	117	64.9	97.6	50.8	76.3	44.4	66.7	35.3	53.1	26.9	40.5

Properties

A_g (in.^{2})	14.7	11.9	9.07	7.85	6.14	4.62
I (in.^{4})	119	100	77.8	68.1	54.1	41.3
r (in.)	2.85	2.89	2.93	2.95	2.97	2.99

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS7.625–
HSS7.500

F_y = 42 ksi

Shape		HSS7.625×				HSS7.500×							
		0.375		0.328		0.500		0.375		0.312		0.250	
t_{design}, in.		0.349		0.305		0.465		0.349		0.291		0.233	
Wt/ft		29.1		25.6		37.4		28.6		24.0		19.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	201	302	176	265	258	388	197	296	166	249	134	201
	6	191	287	168	253	246	369	188	282	158	237	127	192
	7	188	282	165	249	241	362	184	277	155	233	125	188
	8	184	277	162	244	236	355	181	271	152	228	123	185
	9	180	271	159	238	230	346	176	265	148	223	120	180
	10	176	264	155	232	224	337	172	258	145	217	117	176
	11	171	257	150	226	218	327	167	251	141	211	114	171
	12	166	249	146	219	211	316	162	243	136	205	110	166
	13	160	241	141	212	203	305	156	235	132	198	107	160
	14	154	232	136	205	196	294	150	226	127	191	103	155
	15	149	223	131	197	188	282	145	217	122	183	99.0	149
	16	143	214	126	189	180	270	138	208	117	176	95.0	143
	17	136	205	121	181	171	257	132	199	112	168	90.9	137
	18	130	196	115	173	163	245	126	189	107	160	86.7	130
	19	124	186	110	165	155	232	120	180	101	153	82.5	124
	20	118	177	104	156	146	220	114	171	96.2	145	78.3	118
	21	111	167	98.6	148	138	207	107	161	91.0	137	74.1	111
	22	105	158	93.2	140	130	195	101	152	85.9	129	70.0	105
	23	99.1	149	87.8	132	122	183	95.0	143	80.8	121	65.9	99.1
	24	93.1	140	82.6	124	114	171	89.0	134	75.8	114	61.9	93.0
	25	87.2	131	77.4	116	106	160	83.2	125	70.9	107	58.0	87.1
	26	81.5	122	72.4	109	98.6	148	77.6	117	66.1	99.4	54.1	81.4
	27	75.8	114	67.4	101	91.4	137	71.9	108	61.4	92.3	50.3	75.6
	28	70.4	106	62.7	94.2	85.0	128	66.9	101	57.1	85.8	46.8	70.3
	29	65.7	98.7	58.4	87.8	79.3	119	62.4	93.7	53.2	80.0	43.6	65.6
	30	61.4	92.2	54.6	82.0	74.1	111	58.3	87.6	49.7	74.8	40.8	61.3
	32	53.9	81.1	48.0	72.1	65.1	97.8	51.2	77.0	43.7	65.7	35.8	53.9
	34	47.8	71.8	42.5	63.9	57.7	86.7	45.4	68.2	38.7	58.2	31.7	47.7
	36	42.6	64.1	37.9	57.0	51.4	77.3	40.5	60.8	34.5	51.9	28.3	42.6
	38	38.2	57.5	34.0	51.1	46.2	69.4	36.3	54.6	31.0	46.6	25.4	38.2
	40	34.5	51.9	30.7	46.1	41.7	62.6	32.8	49.3	28.0	42.0	22.9	34.5

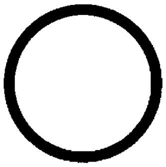
Properties

A_g (in.^{2})	7.98	7.01	10.3	7.84	6.59	5.32
I (in.^{4})	52.9	47.1	63.9	50.2	42.9	35.2
r (in.)	2.58	2.59	2.49	2.53	2.55	2.57

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

F_y = 42 ksi

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS



HSS7.500–
HSS7.000

Shape		HSS7.500×		HSS7.000×							
		0.188		0.500		0.375		0.312		0.250	
t_{design}, in.		0.174		0.465		0.349		0.291		0.233	
Wt/ft		14.7		34.7		26.6		22.3		18.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	101	151	240	361	183	276	154	232	125	187
	6	96.1	144	226	340	173	260	146	219	118	177
	7	94.4	142	221	333	170	255	143	215	116	174
	8	92.6	139	216	325	166	249	140	210	113	170
	9	90.5	136	210	316	161	242	136	204	110	165
	10	88.3	133	204	306	156	235	132	198	107	160
	11	85.9	129	197	296	151	227	128	192	103	155
	12	83.3	125	189	285	146	219	123	185	99.8	150
	13	80.6	121	182	273	140	211	118	178	96.0	144
	14	77.8	117	174	261	134	202	113	170	92.1	138
	15	74.9	113	166	249	128	193	108	163	88.0	132
	16	71.9	108	157	237	122	183	103	155	83.9	126
	17	68.8	103	149	224	116	174	98.0	147	79.8	120
	18	65.7	98.8	141	212	109	164	92.8	139	75.6	114
	19	62.6	94.1	132	199	103	155	87.5	132	71.4	107
	20	59.5	89.4	124	187	96.9	146	82.4	124	67.2	101
	21	56.3	84.7	116	174	90.8	136	77.2	116	63.1	94.8
	22	53.2	80.0	108	162	84.7	127	72.2	108	59.0	88.7
	23	50.2	75.4	100	151	78.9	119	67.3	101	55.1	82.8
	24	47.2	70.9	92.8	140	73.2	110	62.5	93.9	51.2	77.0
	25	44.2	66.4	85.5	129	67.5	101	57.7	86.8	47.4	71.3
	26	41.3	62.1	79.1	119	62.4	93.8	53.4	80.2	43.8	65.9
	27	38.5	57.9	73.3	110	57.9	87.0	49.5	74.4	40.6	61.1
	28	35.8	53.8	68.2	102	53.8	80.9	46.0	69.2	37.8	56.8
	29	33.4	50.1	63.6	95.5	50.2	75.4	42.9	64.5	35.2	53.0
	30	31.2	46.9	59.4	89.3	46.9	70.5	40.1	60.3	32.9	49.5
	32	27.4	41.2	52.2	78.5	41.2	61.9	35.2	53.0	28.9	43.5
	34	24.3	36.5	46.2	69.5	36.5	54.9	31.2	46.9	25.6	38.5
	36	21.7	32.5	41.3	62.0	32.6	48.9	27.8	41.9	22.9	34.4
	38	19.4	29.2	37.0	55.6	29.2	43.9	25.0	37.6	20.5	30.8
	40	17.5	26.4								

Properties

	HSS7.500×0.188	HSS7.000×0.500	HSS7.000×0.375	HSS7.000×0.312	HSS7.000×0.250
A_g (in.^{2})	4.00	9.55	7.29	6.13	4.95
I (in.^{4})	26.9	51.2	40.4	34.6	28.4
r (in.)	2.59	2.32	2.35	2.37	2.39

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS7.000–
HSS6.875

F_y = 42 ksi

Shape		HSS7.000×				HSS6.875×					
		0.188		0.125		0.500		0.375		0.312	
t_{design}, in.		0.174		0.116		0.465		0.349		0.291	
Wt/ft		13.7		9.19		34.1		26.1		21.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	93.8	141	63.1	94.8	236	354	180	270	151	228
	6	88.9	134	59.8	89.9	221	333	170	255	143	215
	7	87.1	131	58.6	88.1	217	325	166	249	140	210
	8	85.2	128	57.3	86.2	211	317	162	243	136	205
	9	83.0	125	55.9	84.0	205	308	157	237	133	199
	10	80.6	121	54.3	81.7	198	298	152	229	129	193
	11	78.1	117	52.7	79.2	191	288	147	221	124	187
	12	75.4	113	50.9	76.5	184	277	142	213	120	180
	13	72.6	109	49.0	73.7	176	265	136	204	115	173
	14	69.7	105	47.1	70.8	168	253	130	195	110	165
	15	66.7	100	45.1	67.8	160	241	124	186	105	158
	16	63.6	95.6	43.1	64.7	152	228	118	177	99.8	150
	17	60.5	91.0	41.0	61.6	144	216	111	168	94.5	142
	18	57.4	86.3	38.9	58.5	135	203	105	158	89.3	134
	19	54.3	81.6	36.8	55.3	127	191	99.0	149	84.1	126
	20	51.1	76.9	34.7	52.2	119	178	92.8	139	78.9	119
	21	48.1	72.2	32.7	49.1	111	166	86.7	130	73.8	111
	22	45.0	67.7	30.6	46.0	103	154	80.7	121	68.8	103
	23	42.0	63.2	28.6	43.1	95.2	143	74.9	113	63.9	96.1
	24	39.2	58.8	26.7	40.1	87.6	132	69.2	104	59.2	89.0
	25	36.3	54.6	24.8	37.3	80.7	121	63.8	95.9	54.6	82.0
	26	33.6	50.5	23.0	34.5	74.7	112	59.0	88.7	50.5	75.8
	27	31.1	46.8	21.3	32.0	69.2	104	54.7	82.2	46.8	70.3
	28	29.0	43.5	19.8	29.7	64.4	96.7	50.9	76.4	43.5	65.4
	29	27.0	40.6	18.4	27.7	60.0	90.2	47.4	71.3	40.6	61.0
	30	25.2	37.9	17.2	25.9	56.1	84.3	44.3	66.6	37.9	57.0
	32	22.2	33.3	15.2	22.8	49.3	74.1	38.9	58.5	33.3	50.1
	34	19.6	29.5	13.4	20.2	43.7	65.6	34.5	51.8	29.5	44.3
	36	17.5	26.3	12.0	18.0	38.9	58.5	30.8	46.2	26.3	39.6
	38	15.7	23.6	10.7	16.2			27.6	41.5	23.6	35.5
	40	14.2	21.3	9.70	14.6						

Properties

	HSS7.000×0.188	HSS7.000×0.125	HSS6.875×0.500	HSS6.875×0.375	HSS6.875×0.312
A_g (in.^{2})	3.73	2.51	9.36	7.16	6.02
I (in.^{4})	21.7	14.9	48.3	38.2	32.7
r (in.)	2.41	2.43	2.27	2.31	2.33

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 42 ksi

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS6.875–
HSS6.625

Shape		HSS6.875×				HSS6.625×					
		0.250		0.188		0.500		0.432		0.375	
t_{design}, in.		0.233		0.174		0.465		0.402		0.349	
Wt/ft		17.7		13.4		32.7		28.6		25.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	122	184	92.1	138	226	340	198	297	173	260
	6	115	173	87.0	131	212	318	185	278	162	244
	7	113	170	85.3	128	207	311	181	272	159	238
	8	110	166	83.3	125	201	302	176	264	154	232
	9	107	161	81.1	122	195	293	171	256	150	225
	10	104	157	78.7	118	188	283	165	248	145	217
	11	101	151	76.1	114	181	272	159	238	139	209
	12	97.1	146	73.4	110	173	260	152	229	134	201
	13	93.3	140	70.6	106	165	249	145	218	128	192
	14	89.3	134	67.7	102	157	237	138	208	122	183
	15	85.3	128	64.6	97.2	149	224	131	197	116	174
	16	81.1	122	61.6	92.5	141	212	124	186	109	164
	17	77.0	116	58.4	87.8	132	199	117	176	103	155
	18	72.8	109	55.3	83.1	124	187	110	165	96.9	146
	19	68.6	103	52.2	78.4	116	174	102	154	90.7	136
	20	64.4	96.8	49.1	73.8	108	162	95.5	143	84.5	127
	21	60.3	90.7	46.0	69.1	99.9	150	88.6	133	78.6	118
	22	56.3	84.6	43.0	64.6	92.3	139	81.9	123	72.7	109
	23	52.4	78.8	40.1	60.2	84.7	127	75.4	113	67.1	101
	24	48.6	73.0	37.2	55.9	77.8	117	69.2	104	61.6	92.6
	25	44.8	67.4	34.4	51.6	71.7	108	63.8	95.9	56.8	85.3
	26	41.4	62.3	31.8	47.7	66.3	99.6	59.0	88.7	52.5	78.9
	27	38.4	57.8	29.5	44.3	61.5	92.4	54.7	82.2	48.7	73.1
	28	35.7	53.7	27.4	41.2	57.2	85.9	50.9	76.4	45.2	68.0
	29	33.3	50.1	25.5	38.4	53.3	80.1	47.4	71.3	42.2	63.4
	30	31.1	46.8	23.9	35.9	49.8	74.8	44.3	66.6	39.4	59.2
	32	27.4	41.1	21.0	31.5	43.8	65.8	38.9	58.5	34.6	52.1
	34	24.2	36.4	18.6	27.9	38.8	58.3	34.5	51.8	30.7	46.1
	36	21.6	32.5	16.6	24.9	34.6	52.0	30.8	46.2	27.4	41.1
	38	19.4	29.2	14.9	22.4						

Properties

	HSS6.875×0.250	HSS6.875×0.188	HSS6.625×0.500	HSS6.625×0.432	HSS6.625×0.375
A_g (in.^{2})	4.86	3.66	9.00	7.86	6.88
I (in.^{4})	26.8	20.6	42.9	38.2	34.0
r (in.)	2.35	2.37	2.18	2.20	2.22

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS6.625

F_y = 42 ksi

Shape		HSS6.625×									
		0.312		0.280		0.250		0.188		0.125	
t_{design}, in.		0.291		0.260		0.233		0.174		0.116	
Wt/ft		21.1		19.0		17.0		12.9		8.69	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	146	219	131	197	118	177	88.7	133	59.7	89.7
	6	137	205	123	185	111	166	83.4	125	56.2	84.4
	7	134	201	120	180	108	162	81.6	123	55.0	82.6
	8	130	196	117	176	105	158	79.5	120	53.6	80.6
	9	126	190	114	171	102	154	77.3	116	52.1	78.3
	10	122	184	110	165	99.0	149	74.8	112	50.5	75.9
	11	118	177	106	159	95.5	143	72.2	109	48.7	73.3
	12	113	170	102	153	91.7	138	69.4	104	46.9	70.5
	13	108	163	97.4	146	87.9	132	66.6	100	45.0	67.6
	14	103	155	92.9	140	83.8	126	63.6	95.5	43.0	64.6
	15	98.0	147	88.3	133	79.7	120	60.5	90.9	41.0	61.6
	16	92.8	139	83.7	126	75.6	114	57.4	86.3	38.9	58.5
	17	87.6	132	79.0	119	71.4	107	54.3	81.6	36.8	55.3
	18	82.3	124	74.3	112	67.2	101	51.1	76.9	34.7	52.2
	19	77.2	116	69.7	105	63.0	94.7	48.0	72.2	32.7	49.1
	20	72.0	108	65.1	97.8	58.9	88.6	44.9	67.6	30.6	46.0
	21	67.0	101	60.6	91.1	54.9	82.5	41.9	63.0	28.6	42.9
	22	62.1	93.4	56.2	84.5	51.0	76.6	39.0	58.6	26.6	40.0
	23	57.4	86.3	52.0	78.1	47.1	70.8	36.1	54.3	24.7	37.1
	24	52.7	79.3	47.8	71.8	43.4	65.2	33.3	50.0	22.8	34.2
	25	48.6	73.0	44.0	66.2	40.0	60.1	30.7	46.1	21.0	31.5
	26	44.9	67.5	40.7	61.2	36.9	55.5	28.3	42.6	19.4	29.2
	27	41.7	62.6	37.8	56.8	34.3	51.5	26.3	39.5	18.0	27.0
	28	38.7	58.2	35.1	52.8	31.9	47.9	24.4	36.7	16.7	25.1
	29	36.1	54.3	32.7	49.2	29.7	44.6	22.8	34.2	15.6	23.4
	30	33.8	50.7	30.6	46.0	27.8	41.7	21.3	32.0	14.6	21.9
	32	29.7	44.6	26.9	40.4	24.4	36.7	18.7	28.1	12.8	19.3
	34	26.3	39.5	23.8	35.8	21.6	32.5	16.6	24.9	11.3	17.1
	36	23.4	35.2	21.2	31.9	19.3	29.0	14.8	22.2	10.1	15.2
	38							13.3	19.9	9.08	13.7

Properties

	0.312	0.280	0.250	0.188	0.125
A_g (in.^{2})	5.79	5.20	4.68	3.53	2.37
I (in.^{4})	29.1	26.4	23.9	18.4	12.6
r (in.)	2.24	2.25	2.26	2.28	2.30

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 42 ksi

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS6.000

Shape		HSS6.000×											
		0.500		0.375		0.312		0.280		0.250		0.188	
t_{design}, in.		0.465		0.349		0.291		0.260		0.233		0.174	
Wt/ft		29.4		22.5		19.0		17.1		15.4		11.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	203	306	156	234	131	197	118	177	106	160	80.1	120
	1	203	305	155	234	131	197	118	177	106	159	79.9	120
	2	201	303	154	232	130	196	117	176	105	158	79.4	119
	3	199	299	153	230	129	193	116	174	104	157	78.6	118
	4	196	295	150	226	127	191	114	171	103	154	77.5	116
	5	192	289	147	222	124	187	112	168	101	151	76.0	114
	6	187	281	144	216	121	182	109	164	98.4	148	74.3	112
	7	182	273	140	210	118	177	106	160	95.7	144	72.3	109
	8	176	264	135	203	114	172	103	155	92.7	139	70.1	105
	9	169	254	130	196	110	166	99.1	149	89.4	134	67.7	102
	10	162	243	125	188	106	159	95.2	143	85.9	129	65.0	97.7
	11	154	232	119	179	101	152	91.0	137	82.1	123	62.3	93.6
	12	146	220	113	170	96.1	144	86.6	130	78.2	118	59.3	89.2
	13	138	207	107	161	91.0	137	82.1	123	74.1	111	56.3	84.7
	14	130	195	101	152	85.9	129	77.5	116	70.0	105	53.3	80.0
	15	121	182	94.8	143	80.6	121	72.8	109	65.8	98.9	50.1	75.3
	16	113	170	88.6	133	75.4	113	68.1	102	61.6	92.6	47.0	70.6
	17	105	158	82.3	124	70.2	106	63.5	95.4	57.5	86.4	43.9	65.9
	18	96.7	145	76.2	115	65.1	97.8	58.9	88.5	53.3	80.2	40.8	61.3
	19	88.9	134	70.2	106	60.1	90.3	54.4	81.8	49.3	74.1	37.8	56.8
	20	81.3	122	64.4	96.9	55.2	83.0	50.0	75.2	45.4	68.2	34.8	52.3
	21	73.8	111	58.8	88.3	50.5	75.8	45.8	68.8	41.6	62.5	32.0	48.0
	22	67.3	101	53.5	80.5	46.0	69.1	41.7	62.7	37.9	57.0	29.2	43.8
	23	61.5	92.5	49.0	73.6	42.1	63.2	38.2	57.4	34.7	52.1	26.7	40.1
	24	56.5	84.9	45.0	67.6	38.6	58.1	35.1	52.7	31.9	47.9	24.5	36.8
	25	52.1	78.3	41.5	62.3	35.6	53.5	32.3	48.6	29.4	44.1	22.6	33.9
	26	48.1	72.4	38.3	57.6	32.9	49.5	29.9	44.9	27.1	40.8	20.9	31.4
	28	41.5	62.4	33.1	49.7	28.4	42.7	25.8	38.7	23.4	35.2	18.0	27.1
	30	36.2	54.4	28.8	43.3	24.7	37.2	22.4	33.7	20.4	30.6	15.7	23.6
	32	31.8	47.8	25.3	38.0	21.7	32.7	19.7	29.6	17.9	26.9	13.8	20.7
	34									15.9	23.9	12.2	18.4

Properties

	0.500	0.375	0.312	0.280	0.250	0.188
A_g (in.^{2})	8.09	6.20	5.22	4.69	4.22	3.18
I (in.^{4})	31.2	24.8	21.3	19.3	17.6	13.5
r (in.)	1.96	2.00	2.02	2.03	2.04	2.06

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

Table 4–5 (continued)

Available Strength in Axial Compression, kips

F_y = 42 ksi

HSS6.000–
HSS5.563

Round HSS

Shape		HSS6.000×		HSS5.563×									
		0.125		0.500		0.375		0.258		0.188		0.134	
t_{design}, in.		0.116		0.465		0.349		0.240		0.174		0.124	
Wt/ft		7.85		27.1		20.8		14.6		10.8		7.78	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	53.9	81.1	187	282	144	216	101	152	74.1	111	53.3	80.1
	1	53.8	80.9	187	281	143	216	101	151	73.9	111	53.2	79.9
	2	53.5	80.4	185	278	142	214	99.9	150	73.4	110	52.8	79.3
	3	52.9	79.6	183	275	140	211	98.7	148	72.5	109	52.2	78.4
	4	52.2	78.4	179	270	138	207	97.0	146	71.3	107	51.3	77.1
	5	51.2	77.0	175	263	135	203	94.8	143	69.7	105	50.2	75.4
	6	50.1	75.3	170	255	131	197	92.3	139	67.9	102	48.9	73.5
	7	48.8	73.3	164	247	127	190	89.3	134	65.8	98.8	47.4	71.2
	8	47.3	71.1	158	237	122	183	86.1	129	63.4	95.3	45.7	68.7
	9	45.7	68.7	151	226	117	175	82.5	124	60.8	91.4	43.9	66.0
	10	44.0	66.1	143	215	111	167	78.7	118	58.1	87.3	42.0	63.1
	11	42.1	63.3	135	203	105	158	74.7	112	55.2	82.9	39.9	60.0
	12	40.2	60.4	127	191	99.0	149	70.5	106	52.2	78.4	37.8	56.8
	13	38.2	57.4	119	178	92.8	139	66.2	99.6	49.1	73.8	35.6	53.5
	14	36.1	54.3	110	166	86.5	130	61.9	93.1	46.0	69.1	33.4	50.1
	15	34.1	51.2	102	153	80.3	121	57.6	86.6	42.8	64.4	31.1	46.8
	16	32.0	48.0	93.8	141	74.1	111	53.3	80.2	39.7	59.7	28.9	43.4
	17	29.9	44.9	85.8	129	68.0	102	49.1	73.8	36.7	55.1	26.7	40.1
	18	27.8	41.8	78.1	117	62.1	93.3	45.0	67.7	33.7	50.6	24.6	36.9
	19	25.8	38.8	70.5	106	56.4	84.8	41.1	61.7	30.8	46.3	22.5	33.8
	20	23.8	35.8	63.7	95.7	50.9	76.5	37.2	55.9	27.9	42.0	20.5	30.7
	21	21.9	32.9	57.7	86.8	46.2	69.4	33.7	50.7	25.3	38.1	18.6	27.9
	22	20.0	30.1	52.6	79.1	42.1	63.3	30.7	46.2	23.1	34.7	16.9	25.4
	23	18.3	27.5	48.1	72.3	38.5	57.9	28.1	42.2	21.1	31.7	15.5	23.2
	24	16.8	25.3	44.2	66.4	35.4	53.1	25.8	38.8	19.4	29.2	14.2	21.4
	25	15.5	23.3	40.7	61.2	32.6	49.0	23.8	35.8	17.9	26.9	13.1	19.7
	26	14.3	21.5	37.7	56.6	30.1	45.3	22.0	33.1	16.5	24.8	12.1	18.2
	28	12.4	18.6	32.5	48.8	26.0	39.0	19.0	28.5	14.3	21.4	10.4	15.7
	30	10.8	16.2	28.3	42.5	22.6	34.0	16.5	24.8	12.4	18.7	9.09	13.7
	32	9.46	14.2									7.99	12.0
	34	8.38	12.6										

Properties

	HSS6.000×0.125	HSS5.563×0.500	HSS5.563×0.375	HSS5.563×0.258	HSS5.563×0.188	HSS5.563×0.134
A_g (in.^{2})	2.14	7.45	5.72	4.01	2.95	2.12
I (in.^{4})	9.28	24.4	19.5	14.2	10.7	7.84
r (in.)	2.08	1.81	1.85	1.88	1.91	1.92

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 42 ksi

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS5.500–
HSS5.000

Shape		HSS5.500×						HSS5.000×					
		0.500		0.375		0.258		0.500		0.375		0.312	
t_{design}, in.		0.465		0.349		0.240		0.465		0.349		0.291	
Wt/ft		26.7		20.5		14.5		24.1		18.5		15.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	185	278	142	213	99.7	150	167	250	128	193	108	163
	1	184	277	142	213	99.5	150	166	250	128	192	108	162
	2	183	275	141	211	98.7	148	164	247	127	190	107	161
	3	180	271	139	208	97.5	147	162	243	125	187	105	158
	4	177	266	136	205	95.7	144	158	237	122	183	103	155
	5	173	259	133	200	93.6	141	153	230	118	178	100	150
	6	167	252	129	194	91.0	137	147	222	114	171	96.6	145
	7	162	243	125	187	88.0	132	141	212	109	164	92.7	139
	8	155	233	120	180	84.7	127	134	201	104	157	88.3	133
	9	148	222	115	172	81.1	122	126	190	98.5	148	83.7	126
	10	140	211	109	164	77.3	116	119	178	92.6	139	78.8	118
	11	132	199	103	155	73.2	110	110	166	86.5	130	73.7	111
	12	124	187	96.9	146	69.1	104	102	153	80.3	121	68.5	103
	13	116	174	90.7	136	64.8	97.4	93.7	141	74.0	111	63.3	95.1
	14	108	162	84.4	127	60.5	90.9	85.5	128	67.8	102	58.1	87.3
	15	99.2	149	78.2	117	56.2	84.4	77.5	116	61.7	92.7	53.0	79.6
	16	91.1	137	72.0	108	51.9	78.0	69.7	105	55.8	83.8	48.0	72.1
	17	83.1	125	66.0	99.1	47.7	71.7	62.2	93.4	50.1	75.3	43.2	64.9
	18	75.5	113	60.1	90.3	43.6	65.6	55.4	83.3	44.7	67.1	38.6	58.0
	19	68.0	102	54.4	81.8	39.7	59.7	49.8	74.8	40.1	60.3	34.6	52.1
	20	61.3	92.2	49.1	73.8	35.9	53.9	44.9	67.5	36.2	54.4	31.3	47.0
	21	55.6	83.6	44.5	66.9	32.5	48.9	40.7	61.2	32.8	49.3	28.4	42.6
	22	50.7	76.2	40.6	61.0	29.6	44.6	37.1	55.8	29.9	44.9	25.8	38.8
	23	46.4	69.7	37.1	55.8	27.1	40.8	34.0	51.0	27.4	41.1	23.6	35.5
	24	42.6	64.0	34.1	51.3	24.9	37.4	31.2	46.9	25.1	37.8	21.7	32.6
	25	39.3	59.0	31.4	47.2	23.0	34.5	28.7	43.2	23.2	34.8	20.0	30.1
	26	36.3	54.6	29.1	43.7	21.2	31.9	26.6	39.9	21.4	32.2	18.5	27.8
	28	31.3	47.0	25.1	37.7	18.3	27.5						
	30			21.8	32.8	15.9	24.0						

Properties

A_g (in.^{2})	7.36	5.65	3.97	6.62	5.10	4.30
I (in.^{4})	23.5	18.8	13.7	17.2	13.9	12.0
r (in.)	1.79	1.83	1.86	1.61	1.65	1.67

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

Table 4–5 (continued)
Available Strength in Axial Compression, kips

F_y = 42 ksi

HSS5.000–
HSS4.500

Round HSS

Shape		HSS5.000×								HSS4.500×			
		0.258		0.250		0.188		0.125		0.375		0.337	
t_{design}, in.		0.240		0.233		0.174		0.116		0.349		0.313	
Wt/ft		13.1		12.7		9.67		6.51		16.5		15.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	90.3	136	87.8	132	66.3	99.7	44.8	67.3	114	172	104	156
	1	90.0	135	87.5	131	66.1	99.4	44.6	67.1	114	171	103	155
	2	89.1	134	86.7	130	65.5	98.5	44.2	66.5	113	169	102	153
	3	87.8	132	85.3	128	64.6	97.0	43.6	65.5	110	166	99.9	150
	4	85.9	129	83.5	126	63.2	95.0	42.7	64.2	107	161	97.1	146
	5	83.5	125	81.2	122	61.5	92.4	41.6	62.5	103	155	93.7	141
	6	80.7	121	78.5	118	59.5	89.4	40.2	60.5	98.8	149	89.6	135
	7	77.5	116	75.4	113	57.2	85.9	38.7	58.2	93.7	141	85.1	128
	8	73.9	111	71.9	108	54.6	82.1	37.0	55.7	88.2	133	80.1	120
	9	70.1	105	68.2	103	51.9	78.0	35.2	52.9	82.3	124	74.8	112
	10	66.1	99.4	64.3	96.7	49.0	73.6	33.3	50.0	76.1	114	69.3	104
	11	61.9	93.1	60.3	90.6	46.0	69.1	31.3	47.0	69.9	105	63.7	95.8
	12	57.6	86.6	56.1	84.3	42.9	64.4	29.2	43.9	63.6	95.6	58.1	87.3
	13	53.3	80.1	51.9	78.0	39.7	59.7	27.1	40.8	57.5	86.4	52.5	79.0
	14	49.0	73.7	47.7	71.8	36.6	55.0	25.0	37.6	51.5	77.4	47.2	70.9
	15	44.8	67.3	43.6	65.6	33.5	50.4	23.0	34.5	45.7	68.7	42.0	63.1
	16	40.7	61.1	39.6	59.6	30.5	45.9	21.0	31.5	40.3	60.5	37.0	55.6
	17	36.7	55.1	35.8	53.8	27.6	41.5	19.0	28.6	35.7	53.6	32.8	49.3
	18	32.8	49.3	32.0	48.1	24.8	37.2	17.1	25.7	31.8	47.8	29.2	43.9
	19	29.5	44.3	28.7	43.2	22.2	33.4	15.4	23.1	28.5	42.9	26.2	39.4
	20	26.6	40.0	25.9	39.0	20.1	30.2	13.9	20.8	25.8	38.7	23.7	35.6
	21	24.1	36.3	23.5	35.3	18.2	27.4	12.6	18.9	23.4	35.1	21.5	32.3
	22	22.0	33.0	21.4	32.2	16.6	24.9	11.5	17.2	21.3	32.0	19.6	29.4
	23	20.1	30.2	19.6	29.5	15.2	22.8	10.5	15.7	19.5	29.3	17.9	26.9
	24	18.5	27.8	18.0	27.1	13.9	20.9	9.62	14.5	17.9	26.9	16.4	24.7
	25	17.0	25.6	16.6	24.9	12.8	19.3	8.87	13.3				
	26	15.7	23.6	15.3	23.1	11.9	17.8	8.20	12.3				
	28	13.6	20.4	13.2	19.9	10.2	15.4	7.07	10.6				

Properties

	0.258	0.250	0.188	0.125	0.375	0.337
A_g (in.^{2})	3.59	3.49	2.64	1.78	4.55	4.12
I (in.^{4})	10.2	9.94	7.69	5.31	9.87	9.07
r (in.)	1.69	1.69	1.71	1.73	1.47	1.48

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 42 ksi

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

HSS4.500–
HSS4.000

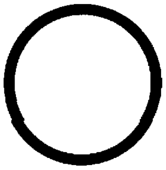
Shape		HSS4.500×						HSS4.000×			
		0.237		0.188		0.125		0.313		0.250	
t_{design}, in.		0.220		0.174		0.116		0.291		0.233	
Wt/ft		10.8		8.67		5.85		12.3		10.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	74.4	112	59.5	89.4	40.2	60.4	85.3	128	69.3	104
	1	74.1	111	59.2	89.1	40.0	60.2	84.8	128	69.0	104
	2	73.3	110	58.6	88.0	39.6	59.5	83.5	126	68.0	102
	3	71.9	108	57.5	86.4	38.9	58.4	81.4	122	66.3	99.7
	4	69.9	105	56.0	84.1	37.9	56.9	78.6	118	64.0	96.3
	5	67.6	102	54.1	81.3	36.6	55.1	75.0	113	61.2	92.1
	6	64.8	97.3	51.9	78.0	35.2	52.9	70.9	107	58.0	87.2
	7	61.6	92.6	49.4	74.3	33.6	50.4	66.4	99.8	54.4	81.7
	8	58.1	87.4	46.7	70.2	31.8	47.7	61.5	92.4	50.5	75.8
	9	54.5	81.8	43.8	65.8	29.8	44.8	56.4	84.7	46.4	69.7
	10	50.6	76.1	40.8	61.3	27.8	41.8	51.1	76.9	42.2	63.4
	11	46.7	70.2	37.7	56.6	25.7	38.7	45.9	69.1	38.0	57.1
	12	42.7	64.2	34.5	51.9	23.7	35.6	40.8	61.4	33.9	51.0
	13	38.8	58.3	31.4	47.2	21.6	32.4	35.9	54.0	30.0	45.0
	14	35.0	52.6	28.4	42.7	19.5	29.4	31.2	47.0	26.1	39.3
	15	31.3	47.0	25.4	38.2	17.6	26.4	27.2	40.9	22.8	34.2
	16	27.7	41.6	22.6	34.0	15.7	23.5	23.9	36.0	20.0	30.1
	17	24.5	36.9	20.0	30.1	13.9	20.9	21.2	31.8	17.7	26.7
	18	21.9	32.9	17.9	26.8	12.4	18.6	18.9	28.4	15.8	23.8
	19	19.6	29.5	16.0	24.1	11.1	16.7	17.0	25.5	14.2	21.3
	20	17.7	26.6	14.5	21.7	10.0	15.1	15.3	23.0	12.8	19.3
	21	16.1	24.2	13.1	19.7	9.09	13.7	13.9	20.9	11.6	17.5
	22	14.6	22.0	11.9	18.0	8.28	12.4	12.7	19.0	10.6	15.9
	23	13.4	20.1	10.9	16.4	7.58	11.4				
	24	12.3	18.5	10.0	15.1	6.96	10.5				
	25	11.3	17.0	9.25	13.9	6.41	9.64				

Properties

	HSS4.500×0.237	HSS4.500×0.188	HSS4.500×0.125	HSS4.000×0.313	HSS4.000×0.250
A_g (in.^{2})	2.96	2.36	1.60	3.39	2.76
I (in.^{4})	6.79	5.54	3.84	5.87	4.91
r (in.)	1.52	1.53	1.55	1.32	1.33

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



HSS4.000

Table 4–5 (continued)
Available Strength in Axial Compression, kips
Round HSS

$F_y = 42$ ksi

Shape		HSS4.000×									
		0.237		0.226		0.220		0.188		0.125	
t_{design}, in.		0.220		0.210		0.205		0.174		0.116	
Wt/ft		9.53		9.12		8.89		7.66		5.18	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y	0	65.7	98.8	62.9	94.5	61.5	92.4	52.6	79.1	35.6	53.5
	1	65.4	98.3	62.6	94.1	61.2	91.9	52.3	78.7	35.4	53.3
	2	64.4	96.8	61.7	92.7	60.3	90.6	51.6	77.5	34.9	52.5
	3	62.8	94.5	60.2	90.4	58.8	88.4	50.4	75.7	34.1	51.3
	4	60.7	91.3	58.1	87.4	56.8	85.4	48.7	73.2	33.0	49.6
	5	58.1	87.3	55.6	83.6	54.4	81.7	46.6	70.1	31.7	47.6
	6	55.0	82.7	52.7	79.2	51.5	77.4	44.2	66.5	30.1	45.2
	7	51.6	77.5	49.4	74.3	48.4	72.7	41.5	62.4	28.3	42.5
	8	47.9	72.0	45.9	69.0	44.9	67.5	38.6	58.1	26.4	39.6
	9	44.1	66.2	42.2	63.5	41.3	62.1	35.6	53.5	24.4	36.6
	10	40.1	60.3	38.5	57.8	37.7	56.6	32.5	48.8	22.3	33.5
	11	36.2	54.4	34.7	52.2	34.0	51.1	29.3	44.1	20.2	30.3
	12	32.3	48.5	31.0	46.6	30.4	45.6	26.3	39.5	18.1	27.2
	13	28.5	42.9	27.4	41.2	26.9	40.4	23.3	35.0	16.1	24.2
	14	24.9	37.5	24.0	36.0	23.5	35.3	20.4	30.7	14.2	21.4
	15	21.7	32.6	20.9	31.4	20.5	30.8	17.8	26.7	12.4	18.6
	16	19.1	28.7	18.4	27.6	18.0	27.0	15.6	23.5	10.9	16.4
	17	16.9	25.4	16.3	24.4	15.9	24.0	13.9	20.8	9.65	14.5
	18	15.1	22.7	14.5	21.8	14.2	21.4	12.4	18.6	8.61	12.9
	19	13.5	20.3	13.0	19.6	12.8	19.2	11.1	16.7	7.72	11.6
	20	12.2	18.4	11.8	17.7	11.5	17.3	10.0	15.0	6.97	10.5
	21	11.1	16.7	10.7	16.0	10.4	15.7	9.08	13.6	6.32	9.50
	22	10.1	15.2	9.71	14.6	9.52	14.3	8.27	12.4	5.76	8.66

Properties

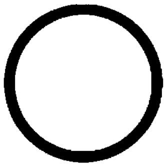
A_g (in.^{2})	2.61	2.50	2.44	2.09	1.42
I (in.^{4})	4.68	4.50	4.41	3.83	2.67
r (in.)	1.34	1.34	1.34	1.35	1.37

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 35 ksi

Table 4–6
Available Strength in Axial Compression, kips
Pipe



PIPE 12–PIPE 8

Shape		Pipe 12				Pipe 10				Pipe 8			
		XS		Std		XS		Std		XXS		XS	
t_{design}, in.		0.465		0.349		0.465		0.340		0.816		0.465	
Wt/ft		65.5		49.6		54.8		40.5		72.5		43.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	376	565	285	428	315	473	233	350	419	630	250	375
	6	371	557	281	422	309	464	229	343	405	609	242	364
	7	369	555	280	420	306	461	227	341	400	601	239	360
	8	367	551	278	418	304	457	225	338	394	593	236	355
	9	364	548	276	415	301	452	223	335	388	583	233	350
	10	362	544	274	412	298	448	221	332	381	573	229	344
	11	359	539	272	409	294	442	218	328	373	561	225	337
	12	356	534	270	405	291	437	215	324	365	549	220	331
	13	352	529	267	401	287	431	213	320	357	536	215	323
	14	348	524	264	397	282	424	209	315	348	522	210	316
	15	345	518	261	393	278	418	206	310	338	508	205	308
	16	340	512	258	388	273	410	203	305	328	493	199	300
	17	336	505	255	383	268	403	199	299	318	478	194	291
	18	331	498	252	378	263	395	195	294	307	462	188	282
	19	327	491	248	373	258	387	192	288	297	446	182	273
	20	322	484	244	367	252	379	187	282	286	430	176	264
	21	317	476	241	362	246	370	183	276	275	413	169	254
	22	311	468	237	356	241	362	179	269	264	397	163	245
	23	306	460	233	350	235	353	175	263	253	380	157	235
	24	300	452	229	343	229	344	170	256	242	363	150	226
	25	295	443	224	337	222	334	166	249	231	347	144	216
	26	289	434	220	331	216	325	161	243	220	330	138	207
	27	283	425	216	324	210	316	157	236	209	314	131	197
	28	277	416	211	317	204	306	152	229	198	298	125	188
	29	271	407	206	310	197	296	148	222	188	282	119	179
	30	265	398	202	303	191	287	143	215	177	266	113	170
	32	252	379	192	289	178	268	134	201	157	236	101	152
	34	240	360	183	275	166	249	124	187	139	209	89.9	135
	36	227	341	173	261	153	230	115	173	124	187	80.2	120
	38	214	322	164	246	141	212	106	160	111	167	71.9	108
	40	201	303	154	232	129	194	97.7	147	101	151	64.9	97.6

Properties

	Pipe 12 XS	Pipe 12 Std	Pipe 10 XS	Pipe 10 Std	Pipe 8 XXS	Pipe 8 XS
A_g (in.^{2})	17.9	13.6	15.0	11.1	20.0	11.9
I (in.^{4})	339	262	199	151	154	100
r (in.)	4.35	4.39	3.64	3.68	2.78	2.89

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–6 (continued)
Available Strength in Axial Compression, kips

F_y = 35 ksi

PIPE 8–PIPE 5

Pipe

Shape		Pipe 8		Pipe 6						Pipe 5	
		Std		XXS		XS		Std		XXS	
t_{design}, in.		0.300		0.805		0.403		0.261		0.699	
Wt/ft		28.6		53.2		28.6		19.0		38.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	165	247	308	463	165	248	109	164	224	337
	6	160	240	290	435	156	235	104	156	205	309
	7	158	237	283	426	153	230	102	153	199	299
	8	156	234	276	415	150	225	99.7	150	192	288
	9	154	231	268	403	146	220	97.3	146	184	277
	10	151	227	260	390	142	213	94.6	142	176	264
	11	148	223	251	377	137	207	91.8	138	167	251
	12	146	219	241	362	133	200	88.7	133	158	237
	13	143	214	231	347	128	192	85.6	129	148	223
	14	139	209	220	331	123	184	82.3	124	139	209
	15	136	204	210	315	117	176	78.9	119	129	195
	16	132	199	199	299	112	168	75.4	113	120	180
	17	129	193	188	283	107	160	71.9	108	111	166
	18	125	188	177	266	101	152	68.3	103	102	153
	19	121	182	166	250	95.5	144	64.7	97.3	92.9	140
	20	117	176	156	234	90.0	135	61.2	91.9	84.3	127
	21	113	170	145	218	84.6	127	57.6	86.6	76.4	115
	22	109	164	135	203	79.3	119	54.1	81.4	69.7	105
	23	105	158	125	188	74.0	111	50.7	76.2	63.7	95.8
	24	101	152	115	173	68.9	104	47.4	71.2	58.5	88.0
	25	96.7	145	106	159	64.0	96.1	44.1	66.3	53.9	81.1
	26	92.6	139	97.9	147	59.1	88.9	40.9	61.4	49.9	75.0
	27	88.6	133	90.8	137	54.8	82.4	37.9	57.0	46.2	69.5
	28	84.5	127	84.5	127	51.0	76.6	35.2	53.0	43.0	64.6
	29	80.5	121	78.7	118	47.5	71.4	32.9	49.4	40.1	60.2
	30	76.6	115	73.6	111	44.4	66.7	30.7	46.1		
	32	68.9	104	64.7	97.2	39.0	58.7	27.0	40.6		
	34	61.5	92.4	57.3	86.1	34.6	52.0	23.9	35.9		
	36	54.8	82.4			30.8	46.4	21.3	32.0		
	38	49.2	74.0								
	40	44.4	66.8								

Properties

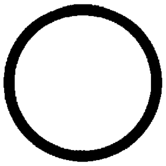
	Pipe 8 Std	Pipe 6 XXS	Pipe 6 XS	Pipe 6 Std	Pipe 5 XXS
A_g (in.^{2})	7.85	14.7	7.88	5.22	10.7
I (in.^{4})	68.1	63.5	38.3	26.5	32.2
r (in.)	2.95	2.08	2.20	2.25	1.74

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 35 ksi

Table 4–6 (continued)
Available Strength in Axial Compression, kips
Pipe



PIPE 5–PIPE 4

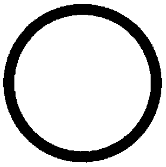
Shape		Pipe 5				Pipe 4					
		XS		Std		XXS		XS		Std	
t_{design}, in.		0.349		0.241		0.628		0.315		0.221	
Wt/ft		20.8		14.6		27.6		15.0		10.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	120	180	84.5	127	160	241	86.8	130	62.2	93.6
	6	111	167	78.4	118	139	210	76.9	116	55.4	83.3
	7	108	162	76.3	115	133	199	73.6	111	53.2	79.9
	8	104	157	73.9	111	125	188	70.0	105	50.7	76.2
	9	101	151	71.4	107	117	176	66.2	99.4	48.0	72.1
	10	96.6	145	68.6	103	109	164	62.1	93.3	45.1	67.9
	11	92.3	139	65.7	98.7	101	151	57.9	87.0	42.2	63.4
	12	87.8	132	62.6	94.1	92.2	139	53.6	80.5	39.2	58.9
	13	83.2	125	59.5	89.4	83.8	126	49.3	74.1	36.2	54.4
	14	78.5	118	56.2	84.5	75.6	114	45.0	67.7	33.2	49.9
	15	73.7	111	52.9	79.5	67.6	102	40.9	61.4	30.2	45.4
	16	69.0	104	49.6	74.6	59.9	90.1	36.8	55.4	27.4	41.1
	17	64.2	96.5	46.3	69.6	53.1	79.8	32.9	49.5	24.6	37.0
	18	59.6	89.5	43.1	64.8	47.3	71.2	29.4	44.1	22.0	33.0
	19	55.0	82.6	39.9	60.0	42.5	63.9	26.4	39.6	19.7	29.6
	20	50.5	76.0	36.8	55.3	38.3	57.6	23.8	35.7	17.8	26.7
	21	46.2	69.5	33.8	50.8	34.8	52.3	21.6	32.4	16.1	24.2
	22	42.1	63.3	30.8	46.3	31.7	47.6	19.7	29.5	14.7	22.1
	23	38.5	57.9	28.2	42.4	29.0	43.6	18.0	27.0	13.4	20.2
	24	35.4	53.2	25.9	38.9			16.5	24.8	12.4	18.6
	25	32.6	49.0	23.9	35.9					11.4	17.1
	26	30.1	45.3	22.1	33.2						
	27	28.0	42.0	20.5	30.8						
	28	26.0	39.1	19.0	28.6						
	29	24.2	36.4	17.7	26.7						
	30	22.6	34.0	16.6	24.9						

Properties

	Pipe 5 XS	Pipe 5 Std	Pipe 4 XXS	Pipe 4 XS	Pipe 4 Std
A_g (in.^{2})	5.72	4.03	7.64	4.14	2.97
I (in.^{4})	19.5	14.3	14.7	9.12	6.82
r (in.)	1.85	1.88	1.39	1.48	1.51

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



PIPE 3½–PIPE 3

Table 4–6 (continued)

Available Strength in Axial Compression, kips

Pipe

F_y = 35 ksi

Shape		Pipe 3½				Pipe 3					
		XS		Std		XXS		XS		Std	
t_{design}, in.		0.296		0.211		0.559		0.280		0.201	
Wt/ft		12.5		9.12		18.6		10.3		7.58	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y	0	72.1	108	52.6	79.1	108	163	59.3	89.1	43.6	65.5
	6	61.8	92.9	45.4	68.2	85.3	128	48.4	72.8	35.9	53.9
	7	58.5	87.9	43.0	64.7	78.3	118	45.0	67.6	33.5	50.3
	8	54.9	82.4	40.5	60.8	71.0	107	41.3	62.1	30.9	46.4
	9	51.0	76.7	37.8	56.7	63.5	95.4	37.5	56.4	28.2	42.3
	10	47.0	70.7	34.9	52.5	56.0	84.2	33.7	50.7	25.4	38.2
	11	43.0	64.6	32.1	48.2	48.8	73.3	30.0	45.0	22.7	34.1
	12	39.0	58.6	29.2	43.8	41.9	63.0	26.3	39.5	20.0	30.1
	13	35.0	52.7	26.3	39.6	35.7	53.7	22.8	34.3	17.5	26.3
	14	31.2	46.9	23.6	35.4	30.8	46.3	19.7	29.6	15.1	22.7
	15	27.5	41.4	20.9	31.5	26.8	40.3	17.1	25.8	13.2	19.8
	16	24.2	36.4	18.4	27.7	23.6	35.4	15.1	22.6	11.6	17.4
	17	21.4	32.2	16.3	24.5	20.9	31.4	13.3	20.1	10.3	15.4
	18	19.1	28.7	14.6	21.9			11.9	17.9	9.15	13.8
	19	17.2	25.8	13.1	19.6			10.7	16.1	8.21	12.3
	20	15.5	23.3	11.8	17.7						
	21	14.1	21.1	10.7	16.1						
	22			9.74	14.6						

Properties

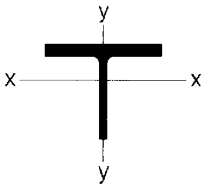
	Pipe 3½ XS	Pipe 3½ Std	Pipe 3 XXS	Pipe 3 XS	Pipe 3 Std
A_g (in.^{2})	3.44	2.51	5.16	2.83	2.08
I (in.^{4})	5.94	4.52	5.79	3.70	2.85
r (in.)	1.31	1.34	1.06	1.14	1.17

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 50 ksi

Table 4-7
Available Strength in Axial Compression, kips
WT Shapes



WT18

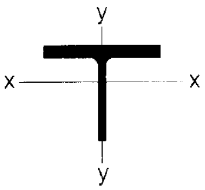
Shape			WT18×									
Wt/ft			151^c		141^c		131^c		123.5^c		115.5^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis	X-X Axis	0	1210	1820	1050	1580	921	1380	813	1220	708	1060
		10	1170	1760	1020	1530	894	1340	791	1190	690	1040
		12	1150	1730	1010	1510	883	1330	782	1180	682	1030
		14	1130	1700	991	1490	869	1310	771	1160	673	1010
		16	1110	1670	972	1460	854	1280	758	1140	663	997
		18	1090	1630	952	1430	837	1260	744	1120	652	980
		20	1060	1590	930	1400	819	1230	729	1100	639	961
		22	1030	1550	906	1360	799	1200	712	1070	626	940
		24	999	1500	880	1320	778	1170	695	1040	611	919
		26	967	1450	853	1280	756	1140	676	1020	596	896
		28	933	1400	825	1240	732	1100	656	986	580	871
		30	897	1350	796	1200	708	1060	635	955	563	846
		32	861	1290	766	1150	682	1030	614	923	545	820
		34	824	1240	735	1100	657	987	592	890	527	793
		36	787	1180	703	1060	630	947	570	857	509	765
		40	712	1070	640	962	577	867	524	788	471	708
	Y-Y Axis	0	1210	1820	1050	1580	921	1380	813	1220	708	1060
		10	1040	1560	902	1360	778	1170	682	1030	589	885
		12	1030	1540	890	1340	769	1160	675	1010	584	877
		14	1000	1510	873	1310	755	1140	665	999	576	865
		16	972	1460	849	1280	737	1110	651	978	565	849
		18	935	1410	820	1230	714	1070	632	950	551	828
		20	893	1340	786	1180	687	1030	610	917	534	803
		22	848	1270	749	1130	657	987	585	880	514	773
		24	800	1200	709	1070	624	938	558	839	492	740
		26	750	1130	668	1000	589	886	529	795	468	704
		28	699	1050	626	940	554	832	499	750	444	667
		30	649	975	583	876	518	778	468	704	418	628
		32	599	900	540	812	481	724	437	657	392	590
		34	549	825	498	749	446	670	406	611	366	550
		36	501	753	457	687	410	617	376	565	340	512
		40	411	618	379	569	342	514	317	476	290	436

Properties

A_g (in.^{2})	44.4	41.5	38.5	36.3	34.0
r_x (in.)	5.37	5.36	5.36	5.36	5.36
r_y (in.)	3.82	3.80	3.76	3.74	3.71

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.



WT18

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

F_y = 50 ksi

Shape			WT18×									
Wt/ft			128^c		116^c		105^c		97^c		91^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	1040	1560	846	1270	731	1100	599	901	511	769
		10	1010	1520	823	1240	713	1070	585	880	501	752
		12	996	1500	813	1220	704	1060	579	871	496	745
		14	980	1470	801	1200	695	1040	572	860	490	737
		16	963	1450	788	1180	684	1030	564	848	484	728
		18	943	1420	773	1160	672	1010	555	835	477	717
		20	922	1390	757	1140	659	990	546	820	470	706
		22	898	1350	740	1110	645	969	535	804	461	693
		24	874	1310	721	1080	629	946	524	787	452	680
		26	848	1270	702	1050	613	921	512	769	443	666
		28	820	1230	681	1020	596	896	499	750	433	651
		30	792	1190	660	992	578	869	486	730	422	635
		32	763	1150	638	958	560	841	472	709	411	618
		34	733	1100	615	924	541	813	457	687	400	601
		36	702	1060	591	889	521	784	443	665	388	583
		40	640	962	544	818	482	724	412	620	364	547
	Y-Y Axis	0	1040	1560	846	1270	731	1100	599	901	511	769
		10	841	1260	684	1030	573	861	472	709	403	605
		12	800	1200	655	984	551	828	456	685	391	588
		14	749	1130	618	928	522	785	436	655	376	564
		16	692	1040	575	865	488	734	411	618	357	536
		18	632	949	530	796	451	678	383	576	335	503
		20	570	856	482	725	412	619	354	531	311	468
		22	508	763	434	652	372	559	323	485	286	431
		24	447	672	387	581	332	499	292	439	261	393
		26	389	585	341	512	294	442	261	393	236	355
		28	338	508	297	446	257	386	232	348	211	318
		30	296	445	260	391	226	339	204	306	187	282
		32	261	393	230	346	200	300	181	272	166	250
		34	232	349	205	308	178	267	161	242	148	223
		36	208	312	183	275	160	240	145	217	133	200
		40	169	254	149	224	130	196	118	178	109	164

Properties

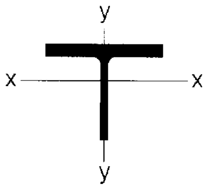
A_g (in.^{2})	37.7	34.1	30.9	28.5	26.8
r_x (in.)	5.66	5.63	5.65	5.62	5.62
r_y (in.)	2.65	2.62	2.58	2.56	2.55

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT18

Shape		WT18×							
Wt/ft		85^c		80^c		75^c		67.5^c	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length KL (ft) with respect to indicated axis: X-X Axis	0	424	637	367	552	322	484	271	408
	10	416	625	361	542	317	476	267	402
	12	412	620	358	538	315	473	265	399
	14	408	614	355	533	312	469	263	396
	16	404	607	351	528	309	464	261	392
	18	398	599	347	522	306	459	258	388
	20	393	590	342	515	302	454	255	384
	22	387	581	337	507	298	447	252	379
	24	380	571	332	499	293	441	249	374
	26	373	560	326	490	289	434	245	368
	28	365	549	320	481	284	426	241	362
	30	357	537	314	472	278	418	237	356
	32	349	525	307	462	273	410	233	350
	34	340	512	300	451	267	401	228	343
	36	331	498	293	440	261	392	223	336
	40	313	470	278	417	248	373	213	321
Y-Y Axis	0	424	637	367	552	322	484	271	408
	10	335	503	288	432	248	373	197	296
	12	327	491	281	423	243	366	193	290
	14	316	474	273	410	236	355	188	282
	16	302	454	262	394	228	342	181	273
	18	286	429	249	374	218	327	174	261
	20	268	402	235	353	206	309	165	248
	22	249	374	219	330	193	290	155	233
	24	229	344	203	305	180	270	144	217
	26	209	315	187	281	166	249	134	201
	28	190	285	170	256	152	228	122	184
	30	171	256	154	232	138	208	111	168
	32	152	228	138	208	125	187	100	151
	34	136	204	124	186	112	168	90.5	136
	36	122	183	111	167	101	151	81.9	123
	40	99.9	150	91.3	137	82.9	125		

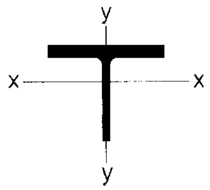
Properties

	85	80	75	67.5
A_g (in.^{2})	25.0	23.5	22.1	19.9
r_x (in.)	5.61	5.61	5.62	5.66
r_y (in.)	2.53	2.50	2.47	2.38

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



WT16.5

Table 4-7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

$F_y = 50$ ksi

Shape		WT16.5×									
Wt/ft		193.5^h		177^h		159		145.5^c		131.5^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis: X-X Axis	0	1710	2560	1560	2340	1400	2110	1270	1910	1050	1580
	10	1640	2460	1500	2250	1340	2020	1220	1830	1010	1520
	12	1610	2420	1470	2210	1320	1980	1200	1800	992	1490
	14	1570	2370	1440	2160	1290	1940	1170	1760	972	1460
	16	1540	2310	1400	2110	1260	1890	1140	1710	949	1430
	18	1490	2250	1360	2050	1220	1840	1110	1670	924	1390
	20	1450	2180	1320	1990	1180	1780	1070	1610	897	1350
	22	1400	2100	1280	1920	1140	1720	1040	1560	868	1300
	24	1350	2020	1230	1850	1100	1650	996	1500	837	1260
	26	1290	1940	1180	1770	1050	1580	954	1430	805	1210
	28	1240	1860	1130	1690	1010	1510	912	1370	772	1160
	30	1180	1770	1070	1610	957	1440	868	1300	738	1110
	32	1120	1680	1020	1530	908	1360	824	1240	703	1060
	34	1060	1600	965	1450	859	1290	779	1170	667	1000
	36	1000	1510	910	1370	809	1220	734	1100	632	950
	40	885	1330	802	1210	711	1070	645	970	561	843
Y-Y Axis	0	1710	2560	1560	2340	1400	2110	1270	1910	1050	1580
	10	1530	2300	1390	2090	1230	1850	1100	1660	903	1360
	12	1480	2230	1340	2020	1200	1800	1080	1620	888	1330
	14	1430	2150	1300	1950	1150	1730	1050	1570	865	1300
	16	1370	2060	1240	1860	1100	1660	1000	1510	835	1260
	18	1300	1960	1180	1780	1050	1580	957	1440	800	1200
	20	1230	1860	1120	1680	993	1490	905	1360	760	1140
	22	1160	1750	1050	1580	933	1400	850	1280	718	1080
	24	1090	1630	983	1480	872	1310	793	1190	673	1010
	26	1010	1520	914	1370	810	1220	735	1100	628	944
	28	936	1410	845	1270	748	1120	677	1020	582	875
	30	860	1290	776	1170	687	1030	620	931	536	806
	32	786	1180	708	1060	626	941	563	847	491	738
	34	714	1070	642	965	567	853	509	765	447	672
	36	644	968	578	869	510	766	457	686	404	608
	40	523	786	470	706	415	623	372	559	330	495

Properties

A_g (in.^{2})	57.0	52.1	46.8	42.8	38.7
r_x (in.)	5.07	5.03	4.99	4.96	4.93
r_y (in.)	3.77	3.74	3.71	3.68	3.65

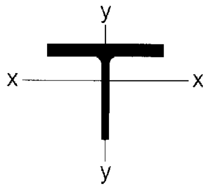
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[c] Shape is slender for compression with $F_y = 50$ ksi.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT16.5

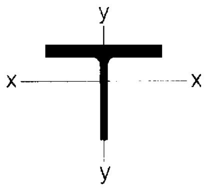
Shape			WT16.5×							
Wt/ft			120.5^c		110.5^c		100.5^c		84.5^c	
Design			P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis	X-X Axis	0	921	1380	782	1170	635	955	465	699
		10	888	1330	755	1130	616	926	454	682
		12	873	1310	744	1120	608	914	449	675
		14	857	1290	731	1100	598	899	443	666
		16	838	1260	716	1080	587	883	436	656
		18	817	1230	699	1050	575	864	429	645
		20	794	1190	681	1020	562	844	421	632
		22	770	1160	662	995	547	823	412	619
		24	744	1120	641	964	532	800	402	605
		26	717	1080	619	931	516	775	392	590
		28	688	1030	597	897	499	750	382	574
		30	659	991	574	862	481	724	371	557
		32	630	946	550	826	463	697	359	540
		34	599	901	525	789	445	669	348	523
		36	569	855	501	752	426	640	336	504
		40	508	764	451	678	388	583	311	467
	Y-Y Axis	0	921	1380	782	1170	635	955	465	699
		10	775	1160	649	975	521	783	382	574
		12	764	1150	641	963	516	775	369	555
		14	747	1120	629	945	508	763	353	530
		16	724	1090	612	920	497	747	334	501
		18	696	1050	591	889	483	726	312	469
		20	664	998	567	852	466	700	289	434
		22	629	945	539	810	446	670	265	398
		24	591	889	509	766	424	638	241	362
		26	553	831	479	719	401	603	217	326
		28	514	772	447	672	377	567	194	291
		30	474	713	415	623	353	530	171	257
		32	436	655	383	576	328	493	151	228
		34	398	598	352	529	304	457	135	203
		36	361	543	321	483	280	420	121	182
		40	295	443	264	397	233	351	98.5	148

Properties

	120.5	110.5	100.5	84.5
A_g (in.^{2})	35.5	32.6	29.6	24.8
r_x (in.)	4.96	4.95	4.95	5.12
r_y (in.)	3.62	3.59	3.56	2.50

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.



WT16.5

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

F_y = 50 ksi

Shape		WT16.5×							
Wt/ft		76^c		70.5^c		65^c		59^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis — X-X Axis	0	385	579	329	494	282	424	233	350
	10	377	566	322	484	276	416	229	344
	12	373	560	319	479	274	412	227	341
	14	368	554	315	474	271	408	225	338
	16	363	546	311	468	268	403	222	334
	18	358	538	307	461	265	398	220	330
	20	352	529	302	454	261	392	217	326
	22	345	518	297	446	257	386	214	321
	24	338	508	291	438	252	379	210	316
	26	330	496	285	429	247	372	207	311
	28	322	484	279	419	242	364	203	305
	30	314	471	272	409	237	356	199	299
	32	305	458	265	399	231	348	195	292
	34	296	445	258	388	225	339	190	286
	36	286	430	250	376	219	330	186	279
	40	267	402	235	353	207	311	176	265
Y-Y Axis	0	385	579	329	494	282	424	233	350
	10	308	463	259	389	216	324	171	257
	12	299	450	252	379	211	317	168	252
	14	288	433	244	366	204	307	163	245
	16	274	412	233	350	196	295	157	236
	18	258	388	221	332	186	280	150	225
	20	240	361	207	311	175	264	142	213
	22	222	334	192	289	164	246	133	200
	24	203	305	177	266	151	227	124	186
	26	184	277	161	243	139	208	114	171
	28	166	249	146	220	126	189	104	156
	30	148	222	131	197	114	171	94.4	142
	32	131	197	117	176	101	153	84.7	127
	34	117	176	104	157	90.9	137	76.2	115
	36	105	158	93.8	141	81.9	123	68.8	103
	40	85.8	129	76.9	116				

Properties

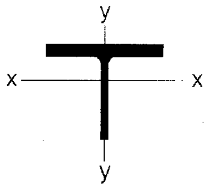
	76	70.5	65	59
A_g (in.^{2})	22.4	20.8	19.2	17.3
r_x (in.)	5.14	5.15	5.18	5.20
r_y (in.)	2.47	2.43	2.38	2.32

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT15

Shape			WT15×											
Wt/ft			195.5^h		178.5^h		163^h		146		130.5		117.5^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to indicated axis	**X-X Axis**	**0**	1720	2590	1570	2360	1430	2160	1290	1930	1150	1730	989	1490
		10	1640	2460	1490	2240	1360	2050	1220	1830	1090	1640	939	1410
		12	1600	2410	1460	2200	1330	2000	1190	1790	1070	1600	918	1380
		14	1560	2350	1420	2140	1300	1950	1160	1740	1040	1560	894	1340
		16	1520	2280	1380	2070	1260	1890	1120	1690	1000	1510	866	1300
		18	1470	2210	1330	2000	1210	1820	1080	1630	969	1460	837	1260
		20	1410	2120	1280	1930	1170	1750	1040	1570	931	1400	804	1210
		22	1360	2040	1230	1850	1120	1680	997	1500	891	1340	770	1160
		24	1290	1950	1170	1760	1070	1600	950	1430	848	1280	735	1100
		26	1230	1850	1120	1680	1010	1520	901	1350	805	1210	698	1050
		28	1170	1760	1060	1590	958	1440	852	1280	760	1140	660	992
		30	1100	1660	996	1500	903	1360	802	1200	715	1070	622	934
		32	1040	1560	936	1410	847	1270	751	1130	669	1010	583	876
		34	971	1460	875	1320	792	1190	701	1050	624	938	545	819
		36	906	1360	816	1230	737	1110	651	979	580	871	507	762
		40	779	1170	699	1050	630	947	555	834	494	742	433	651
	Y-Y Axis	**0**	1720	2590	1570	2360	1430	2160	1290	1930	1150	1730	989	1490
		10	1560	2340	1410	2120	1280	1920	1140	1710	1000	1510	854	1280
		12	1510	2260	1360	2050	1240	1860	1100	1650	970	1460	835	1260
		14	1450	2170	1310	1970	1190	1790	1050	1590	931	1400	808	1210
		16	1380	2080	1250	1880	1130	1700	1010	1510	887	1330	774	1160
		18	1310	1970	1190	1780	1070	1610	953	1430	840	1260	735	1110
		20	1240	1860	1120	1680	1010	1520	897	1350	790	1190	693	1040
		22	1160	1740	1050	1570	947	1420	839	1260	738	1110	648	974
		24	1080	1620	975	1470	880	1320	780	1170	685	1030	602	905
		26	*1000*	*1500*	*902*	*1360*	*813*	*1220*	*719*	*1080*	632	949	*556*	*835*
		28	920	1380	829	1250	746	1120	660	991	578	869	509	765
		30	841	1260	757	1140	680	1020	601	903	526	790	464	697
		32	764	1150	686	1030	616	926	544	817	475	714	419	630
		34	690	1040	619	930	555	834	488	733	425	639	376	565
		36	618	928	553	831	495	745	436	655	380	571	336	506
		40	501	753	449	675	402	604	354	532	309	464	274	412

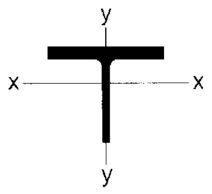
Properties

	195.5	178.5	163	146	130.5	117.5
A_g (in.^{2})	57.6	52.5	47.9	42.9	38.4	34.6
r_x (in.)	4.61	4.56	4.52	4.48	4.46	4.41
r_y (in.)	3.67	3.64	3.60	3.58	3.53	3.51

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

[c] Shape is slender for compression with F_y = 50 ksi.



WT15

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

F_y = 50 ksi

Shape		WT15×									
Wt/ft		105.5^c		95.5^c		86.5^c		74^c		66^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis — X-X Axis	0	837	1260	687	1030	560	841	466	700	385	579
	10	797	1200	658	988	538	809	450	676	373	561
	12	781	1170	645	969	529	795	443	665	368	553
	14	761	1140	630	947	518	778	435	653	362	544
	16	740	1110	614	923	506	760	426	640	355	533
	18	716	1080	596	896	492	740	416	625	347	522
	20	690	1040	576	866	478	718	405	608	339	509
	22	663	996	555	835	462	695	393	591	330	496
	24	634	953	533	802	446	670	380	572	320	481
	26	604	908	510	767	428	644	367	552	310	466
	28	573	862	487	731	410	617	354	532	299	450
	30	542	815	462	695	392	589	340	510	288	434
	32	511	767	438	658	373	561	325	489	277	417
	34	479	720	413	621	354	532	310	466	266	399
	36	448	673	388	584	335	504	295	444	254	382
	40	387	581	340	511	297	446	266	399	230	346
Y-Y Axis	0	837	1260	687	1030	560	841	466	700	385	579
	10	707	1060	574	863	462	694	372	559	298	448
	12	694	1040	566	850	456	686	353	531	285	428
	14	676	1020	553	832	448	673	330	496	268	403
	16	651	979	536	806	437	656	304	457	249	374
	18	621	934	515	774	422	634	277	416	228	343
	20	588	884	491	737	405	608	248	373	206	310
	22	553	830	464	697	385	579	220	331	184	277
	24	515	775	435	654	364	547	193	291	162	244
	26	478	718	406	610	342	514	167	251	141	212
	28	440	661	376	566	319	479	145	218	123	185
	30	402	604	347	521	296	445	127	191	108	163
	32	365	549	317	477	273	411	112	169	95.6	144
	34	330	496	289	434	251	377	99.8	150	85.2	128
	36	296	444	261	392	229	344	89.3	134	76.3	115
	40	241	362	213	320	188	283				

Properties

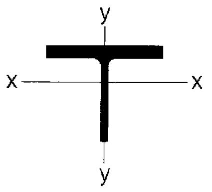
	105.5	95.5	86.5	74	66
A_g (in.^{2})	31.1	28.1	25.5	21.7	19.4
r_x (in.)	4.43	4.42	4.42	4.63	4.66
r_y (in.)	3.49	3.46	3.42	2.28	2.25

ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

^{c} Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT15

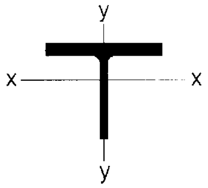
Shape			WT15×									
Wt/ft			62^c		58^c		54^c		49.5^c		45^c	
Design			P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	329	494	290	436	254	381	214	322	160	240
		10	319	480	283	425	247	371	209	315	157	236
		12	315	473	279	420	244	367	207	311	156	234
		14	310	466	275	414	241	362	205	308	154	231
		16	305	458	271	407	237	357	202	303	152	229
		18	299	449	266	399	233	351	199	299	150	226
		20	292	439	260	391	229	344	195	293	148	222
		22	285	429	254	382	224	337	191	288	146	219
		24	278	417	248	373	219	329	187	281	143	215
		26	270	405	241	363	213	321	183	275	140	211
		28	261	393	234	352	208	312	178	268	137	206
		30	253	380	227	341	201	303	174	261	134	202
		32	244	366	219	330	195	293	169	253	131	197
		34	234	352	212	318	189	284	164	246	128	192
		36	225	338	204	306	182	274	158	238	124	187
		40	206	309	187	282	168	253	147	222	117	176
	Y-Y Axis	0	329	494	290	436	254	381	214	322	160	240
		10	254	382	220	331	186	280	152	229	115	173
		12	244	367	212	319	180	271	147	222	113	169
		14	231	348	202	304	172	258	141	212	109	164
		16	216	325	190	285	162	243	134	201	104	157
		18	200	300	176	264	151	226	125	188	99.1	149
		20	182	274	161	242	138	208	116	174	93.0	140
		22	164	247	145	219	126	189	106	159	86.4	130
		24	146	220	130	196	113	170	95.3	143	79.5	119
		26	129	194	115	173	100	151	85.1	128	72.4	109
		28	113	169	101	152	88.0	132	75.1	113	65.4	98.4
		30	98.9	149	88.9	134	77.7	117	66.6	100	58.5	87.9
		32	87.6	132	78.8	118	69.1	104	59.4	89.2	52.3	78.6
		34	78.1	117	70.3	106	61.8	92.8	53.2	79.9	47.0	70.6
		36	70.0	105	63.1	94.9						

Properties

	62	58	54	49.5	45
A_g (in.^{2})	18.2	17.1	15.9	14.5	13.2
r_x (in.)	4.66	4.67	4.69	4.71	4.69
r_y (in.)	2.23	2.19	2.15	2.10	2.09

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^c Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



WT13.5

Table 4-7 (continued)

Available Strength in Axial Compression, kips

WT Shapes

$F_y = 50$ ksi

Shape			WT13.5×											
Wt/ft			129		117.5		108.5		97^c		89^c		80.5^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	1140	1710	1040	1560	958	1440	823	1240	737	1110	607	912
		10	1070	1600	973	1460	896	1350	771	1160	693	1040	573	861
		12	1040	1560	945	1420	870	1310	749	1130	674	1010	559	839
		14	1000	1500	913	1370	840	1260	724	1090	652	980	542	815
		16	962	1450	878	1320	807	1210	696	1050	628	944	524	787
		18	921	1380	839	1260	771	1160	666	1000	602	905	504	757
		20	876	1320	799	1200	733	1100	634	953	574	863	482	725
		22	829	1250	756	1140	692	1040	600	902	545	819	460	691
		24	781	1170	711	1070	651	979	565	850	514	773	436	655
		26	732	1100	666	1000	609	915	530	796	483	726	412	619
		28	682	1030	621	933	566	851	494	742	451	679	387	582
		30	632	950	575	864	524	787	458	688	420	631	362	544
		32	583	877	530	797	482	725	422	635	389	584	337	507
		34	535	804	486	730	441	663	388	583	358	538	313	470
		36	488	734	443	666	402	604	354	532	328	493	289	434
		40	400	601	363	545	328	493	290	436	270	406	242	364
	Y-Y Axis	0	1140	1710	1040	1560	958	1440	823	1240	737	1110	607	912
		10	1000	1510	907	1360	832	1250	705	1060	616	927	503	756
		12	964	1450	872	1310	799	1200	686	1030	602	905	494	742
		14	920	1380	831	1250	762	1150	659	991	581	873	479	720
		16	872	1310	787	1180	722	1080	626	941	553	832	460	691
		18	820	1230	740	1110	678	1020	590	886	522	785	437	657
		20	765	1150	690	1040	632	950	550	827	488	733	411	618
		22	709	1070	638	959	585	879	509	766	452	680	384	577
		24	652	980	586	881	537	808	468	703	416	625	355	534
		26	595	895	534	803	490	736	426	641	379	570	327	491
		28	539	811	483	727	443	665	386	580	343	516	298	448
		30	485	729	434	652	397	597	346	520	308	463	270	406
		32	432	650	386	580	353	530	308	462	274	412	243	365
		34	384	577	342	514	313	471	273	411	244	366	217	326
		36	343	515	306	460	280	421	244	367	218	328	194	292
		40	278	418	248	373	227	342	199	299	178	267	158	238

Properties

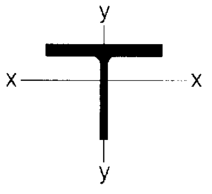
	129	117.5	108.5	97^c	89^c	80.5^c
A_g (in.^{2})	38.0	34.7	32.0	28.6	26.2	23.8
r_x (in.)	4.02	4.00	3.96	3.94	3.97	3.95
r_y (in.)	3.36	3.33	3.32	3.29	3.25	3.23

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with $F_y = 50$ ksi.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT13.5

Shape		WT13.5×											
Wt/ft		73^c		64.5^c		57^c		51^c		47^c		42^c	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis — X-X Axis	0	493	742	432	650	350	526	260	391	219	330	176	264
	10	469	704	412	620	336	504	251	377	212	319	171	257
	12	458	689	404	607	329	495	247	371	209	315	169	254
	14	446	670	394	592	322	484	242	364	206	310	166	250
	16	432	650	383	576	314	472	237	357	202	304	163	246
	18	417	627	371	558	305	459	232	348	198	297	160	241
	20	401	603	358	538	295	444	226	339	193	290	157	236
	22	384	578	344	517	285	428	219	329	188	282	153	230
	24	366	551	330	496	274	412	212	318	182	274	149	224
	26	348	523	315	473	263	395	204	307	176	265	145	218
	28	329	495	299	449	251	377	197	296	170	256	141	211
	30	310	466	283	426	239	359	189	284	164	247	136	204
	32	291	437	267	401	226	340	181	272	158	237	131	197
	34	272	408	251	377	214	322	172	259	151	227	126	190
	36	253	380	235	353	202	303	164	247	144	217	121	182
	40	216	325	204	306	177	266	147	221	131	197	111	167
Y-Y Axis	0	493	742	432	650	350	526	260	391	219	330	176	264
	10	406	610	341	513	270	406	203	305	168	253	130	196
	12	399	600	321	482	256	385	195	292	162	244	126	190
	14	390	586	296	445	239	359	184	277	154	232	121	182
	16	377	566	269	405	219	329	172	258	145	218	114	172
	18	361	542	242	363	198	297	158	237	134	201	107	161
	20	342	514	213	321	176	265	143	216	123	184	98.5	148
	22	322	483	186	280	155	233	129	194	111	167	89.8	135
	24	300	451	160	240	135	202	114	172	99.2	149	81.0	122
	26	278	418	137	206	116	174	100	151	87.8	132	72.3	109
	28	256	385	119	179	101	152	87.5	131	76.8	115	63.8	95.8
	30	234	352	104	156	88.4	133	76.8	115	67.6	102	56.3	84.7
	32	213	320	91.7	138	78.1	117	68.0	102	59.9	90.0	50.1	75.2
	34	192	288	81.5	122	69.5	104	60.5	91.0	53.4	80.3	44.7	67.3
	36	172	259	72.9	110	62.2	93.5						
	40	140	211										

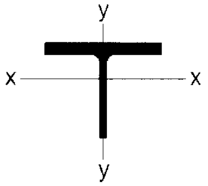
Properties

	73	64.5	57	51	47	42
A_g (in.^{2})	21.6	18.9	16.8	15.0	13.8	12.4
r_x (in.)	3.95	4.13	4.15	4.14	4.16	4.18
r_y (in.)	3.20	2.21	2.18	2.15	2.12	2.07

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



WT12

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

***F*$_y$ = 50 ksi**

Shape			WT12×											
Wt/ft			185^h		167.5^h		153^h		139.5^h		125		114.5	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	1630	2450	1470	2210	1340	2020	1230	1840	1100	1650	1010	1510
		10	1510	2270	1370	2050	1240	1870	1130	1700	1010	1530	927	1390
		12	1470	2200	1320	1990	1200	1810	1100	1650	979	1470	894	1340
		14	1410	2120	1270	1910	1150	1740	1050	1580	939	1410	857	1290
		16	1350	2030	1210	1820	1100	1660	1000	1510	894	1340	816	1230
		18	1280	1930	1150	1730	1050	1570	950	1430	847	1270	771	1160
		20	1210	1820	1090	1640	986	1480	895	1340	796	1200	725	1090
		22	1140	1710	1020	1540	924	1390	837	1260	744	1120	676	1020
		24	1070	1600	953	1430	861	1290	778	1170	690	1040	627	942
		26	991	1490	884	1330	796	1200	719	1080	637	957	578	868
		28	915	1380	815	1220	733	1100	660	993	583	877	528	794
		30	840	1260	746	1120	670	1010	603	906	531	798	480	722
		32	767	1150	679	1020	608	914	546	821	480	722	434	652
		34	696	1050	615	924	549	826	492	740	431	648	389	585
		36	627	943	552	830	492	739	440	661	385	578	347	521
		40	508	764	447	672	398	599	356	535	312	468	281	422
	Y-Y Axis	0	1630	2450	1470	2210	1340	2020	1230	1840	1100	1650	1010	1510
		10	1460	2190	1310	1980	1190	1790	1090	1630	968	1450	880	1320
		12	1400	2100	1260	1890	1140	1720	1040	1560	924	1390	840	1260
		14	1330	2000	1190	1800	1080	1630	983	1480	875	1320	795	1190
		16	1250	1880	1130	1690	1020	1530	924	1390	822	1240	746	1120
		18	1170	1760	1050	1580	952	1430	862	1300	766	1150	694	1040
		20	1090	1640	974	1460	881	1320	797	1200	707	1060	640	962
		22	1000	1510	896	1350	809	1220	730	1100	647	972	585	879
		24	917	1380	817	1230	736	1110	664	998	587	883	530	797
		26	831	1250	739	1110	665	1000	599	900	528	794	476	716
		28	748	1120	663	997	596	895	535	804	471	708	424	638
		30	667	1000	591	888	529	795	474	712	416	626	374	562
		32	590	886	521	782	466	700	417	627	366	551	329	495
		34	523	786	461	693	413	621	370	556	325	488	292	439
		36	466	701	412	619	368	554	330	496	290	436	260	392
		40	378	568	334	502	299	449	267	402	235	353	211	318

Properties

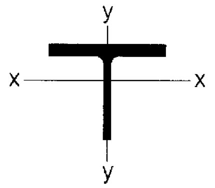
A_g (in.^{2})	54.4	49.2	44.9	41.0	36.8	33.6
r_x (in.)	3.78	3.73	3.69	3.65	3.61	3.58
r_y (in.)	3.27	3.23	3.20	3.17	3.14	3.11

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT12

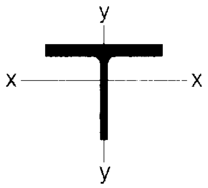
Shape			WT12×									
Wt/ft			103.5		96		88		81		73^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis	X-X Axis	0	909	1370	843	1270	774	1160	715	1070	609	915
		10	836	1260	774	1160	710	1070	656	986	561	844
		12	806	1210	746	1120	684	1030	632	949	542	814
		14	771	1160	714	1070	654	984	604	908	519	780
		16	734	1100	678	1020	622	934	574	862	495	743
		18	693	1040	641	963	587	882	541	814	468	703
		20	651	978	601	903	550	826	507	762	440	661
		22	606	912	559	841	512	769	472	709	411	618
		24	562	844	517	778	473	711	436	655	381	573
		26	517	776	475	715	434	653	400	602	352	529
		28	472	709	434	652	396	595	365	548	322	484
		30	428	644	393	591	359	539	330	497	293	441
		32	386	581	354	532	323	485	297	446	265	398
		34	345	519	316	475	288	432	265	398	238	358
		36	308	463	282	424	257	386	236	355	212	319
		40	250	375	228	343	208	312	191	287	172	259
	Y-Y Axis	0	909	1370	843	1270	774	1160	715	1070	609	915
		10	788	1180	727	1090	661	993	604	907	507	761
		12	752	1130	693	1040	630	947	576	866	491	738
		14	711	1070	655	985	596	895	545	819	469	704
		16	667	1000	614	923	558	839	511	768	442	664
		18	620	932	571	857	518	779	475	714	411	618
		20	571	858	525	790	477	717	437	657	380	570
		22	522	784	479	720	435	653	399	600	347	521
		24	472	709	433	651	393	590	361	542	314	472
		26	423	636	388	584	352	528	323	486	282	423
		28	376	566	345	518	312	469	287	431	250	376
		30	331	497	303	455	273	411	252	378	220	331
		32	291	438	267	401	241	362	222	333	194	292
		34	258	388	237	356	214	321	197	296	173	259
		36	231	347	211	318	191	287	176	264	154	232
		40	187	281	171	258	155	233	143	215	125	189

Properties

	103.5	96	88	81	73^c
A_g (in.^{2})	30.4	28.1	25.8	23.9	21.5
r_x (in.)	3.55	3.53	3.51	3.50	3.50
r_y (in.)	3.08	3.07	3.04	3.05	3.01

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.



WT12

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

F_y = 50 ksi

Shape			WT12×									
Wt/ft			65.5^c		58.5^c		52^c		51.5^c		47^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis	X-X Axis	0	511	768	408	613	317	476	354	532	296	445
		10	474	712	381	573	299	449	333	500	280	421
		12	458	689	370	556	291	438	324	487	273	411
		14	441	662	357	537	282	424	314	472	266	399
		16	421	633	343	516	273	410	303	455	257	386
		18	400	601	328	493	262	394	290	436	247	372
		20	378	568	311	468	250	376	277	417	237	356
		22	355	533	294	442	238	358	263	396	226	340
		24	331	497	276	415	226	339	249	374	215	323
		26	307	461	258	388	213	320	234	352	203	305
		28	283	425	240	361	200	300	219	330	191	287
		30	259	389	222	334	186	280	204	307	179	269
		32	236	355	204	307	173	260	189	285	167	251
		34	214	321	187	281	160	241	175	263	155	233
		36	192	288	170	255	148	222	160	241	144	216
		40	155	234	138	208	123	185	133	200	121	182
	Y-Y Axis	0	511	768	408	613	317	476	354	532	296	445
		10	415	624	326	490	250	375	269	404	225	338
		12	404	608	320	480	246	369	247	372	210	315
		14	389	584	310	466	240	360	223	335	191	287
		16	368	554	296	445	232	348	198	297	171	257
		18	345	519	280	421	221	333	172	258	151	226
		20	320	481	262	394	209	315	147	221	131	196
		22	294	441	243	365	196	295	123	185	111	167
		24	267	401	223	335	182	273	104	157	94.3	142
		26	241	362	203	305	167	252	89.5	134	80.9	122
		28	215	323	183	275	153	230	77.5	116	70.2	105
		30	190	285	164	246	139	209	67.7	102	61.4	92.3
		32	168	252	145	218	125	188	59.7	89.7	54.2	81.4
		34	149	224	129	194	112	168				
		36	134	201	116	174	100	150				
		40	109	163	94.5	142	81.8	123				

Properties

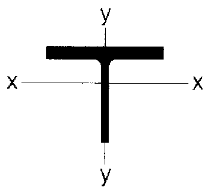
	65.5	58.5	52	51.5	47
A_g (in.^{2})	19.3	17.2	15.3	15.1	13.8
r_x (in.)	3.52	3.51	3.51	3.67	3.67
r_y (in.)	2.97	2.94	2.91	1.99	1.98

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT12

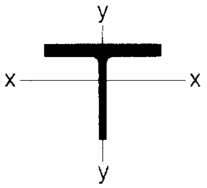
Shape		WT12×									
Wt/ft		42^c		38^c		34^c		31^c		27.5^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis: X-X Axis	0	225	338	181	272	147	221	143	215	109	164
	10	215	323	174	261	142	213	138	207	105	159
	12	210	316	171	256	139	209	135	204	104	156
	14	205	308	167	251	137	205	133	200	102	154
	16	199	299	163	245	134	201	130	195	100	151
	18	193	290	158	238	130	196	126	190	98.0	147
	20	186	280	153	230	126	190	123	185	95.6	144
	22	179	269	148	222	123	184	119	179	93.0	140
	24	171	257	142	214	118	178	115	173	90.2	136
	26	163	245	136	205	114	171	110	166	87.3	131
	28	155	233	130	196	109	165	106	159	84.2	127
	30	147	220	124	186	105	157	101	152	81.1	122
	32	138	208	118	177	100	150	96.7	145	77.9	117
	34	130	195	111	167	95.2	143	91.9	138	74.6	112
	36	121	182	105	158	90.3	136	87.1	131	71.2	107
	40	105	158	92.4	139	80.5	121	77.5	116	64.4	96.8
Y-Y Axis	0	225	338	181	272	147	221	143	215	109	164
	10	172	259	137	206	108	162	90.4	136	68.3	103
	12	162	244	130	196	103	155	80.9	122	62.1	93.3
	14	150	226	122	183	97.0	146	70.3	106	55.0	82.6
	16	137	205	112	168	89.9	135	59.5	89.4	47.4	71.3
	18	122	184	101	152	82.1	123	49.0	73.7	39.8	59.9
	20	108	163	90.6	136	73.9	111	40.7	61.2	33.4	50.1
	22	94.1	141	79.8	120	65.6	98.6	34.3	51.5	28.2	42.4
	24	80.6	121	69.3	104	57.5	86.5				
	26	69.4	104	59.8	89.8	49.8	74.9				
	28	60.2	90.5	52.0	78.2	43.5	65.5				
	30	52.8	79.3	45.7	68.6	38.3	57.6				
	32	46.6	70.1	40.4	60.7						

Properties

	42^c	38^c	34^c	31^c	27.5^c
A_g (in.^{2})	12.4	11.2	10.00	9.11	8.10
r_x (in.)	3.67	3.68	3.70	3.79	3.80
r_y (in.)	1.95	1.92	1.87	1.38	1.34

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



WT10.5

Table 4–7 (continued)

Available Strength in Axial Compression, kips

WT Shapes

F_y = 50 ksi

Shape			WT10.5×											
Wt/ft			100.5		91		83		73.5		66		61^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	886	1330	803	1210	731	1100	647	972	581	873	534	802
		10	794	1190	718	1080	652	980	579	870	519	780	477	716
		12	757	1140	684	1030	620	932	551	828	494	742	453	682
		14	715	1080	645	970	585	879	520	782	466	700	427	642
		16	670	1010	604	907	546	821	487	731	435	654	399	600
		18	622	935	560	841	505	759	451	678	403	606	370	556
		20	572	860	514	773	463	696	414	623	370	556	339	510
		22	522	785	468	704	421	633	377	567	337	506	308	463
		24	472	710	423	635	379	570	341	512	303	456	278	417
		26	423	636	378	568	338	509	305	458	271	407	248	373
		28	376	565	335	504	299	450	270	406	240	361	219	330
		30	331	497	294	442	262	393	237	356	210	316	192	289
		32	291	437	258	388	230	346	208	313	185	278	169	254
		34	257	387	229	344	204	306	185	277	164	246	149	225
		36	230	345	204	307	182	273	165	247	146	219	133	200
		40	186	280	165	249	147	221	133	200	118	178	108	162
	Y-Y Axis	0	886	1330	803	1210	731	1100	647	972	581	873	534	802
		10	774	1160	697	1050	632	949	548	824	486	730	439	659
		12	737	1110	663	997	601	903	521	783	462	694	423	636
		14	695	1040	625	939	566	850	491	738	435	654	401	603
		16	649	976	584	877	528	794	458	688	406	610	375	563
		18	601	904	540	811	488	734	423	636	375	563	346	520
		20	552	829	495	744	447	672	387	582	343	515	316	475
		22	502	754	449	675	406	610	351	527	310	466	285	429
		24	452	679	404	607	365	548	315	473	278	418	255	383
		26	403	606	360	541	324	488	280	420	247	371	226	339
		28	356	536	318	477	286	430	246	370	217	326	197	296
		30	312	468	278	417	250	375	215	323	189	285	172	259
		32	274	412	244	367	220	330	189	284	167	251	152	229
		34	243	365	216	325	195	293	168	252	148	223	135	203
		36	217	326	193	290	174	262	150	225	132	199	121	181
		40	176	264	157	236	141	212	122	183	107	161	98.1	147

Properties

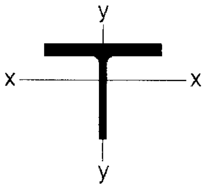
	100.5	91	83	73.5	66	61^c
A_g (in.^{2})	29.6	26.8	24.4	21.6	19.4	17.9
r_x (in.)	3.10	3.07	3.04	3.08	3.06	3.04
r_y (in.)	3.02	3.00	2.99	2.95	2.93	2.91

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT10.5

Shape			WT10.5×											
Wt/ft			55.5^c		50.5^c		46.5^c		41.5^c		36.5^c		34^c	
Design			P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	450	676	369	554	395	594	312	469	234	352	200	300
		10	405	608	335	504	359	539	286	430	217	327	187	280
		12	386	580	321	483	344	517	275	414	210	316	181	272
		14	366	549	305	459	327	492	263	396	202	304	175	263
		16	343	516	288	433	309	464	250	376	193	291	168	252
		18	319	480	270	406	289	435	236	354	184	276	160	241
		20	295	443	251	377	269	404	221	332	174	261	152	228
		22	270	405	231	348	248	372	205	308	163	245	144	216
		24	245	368	212	318	227	341	189	285	152	229	135	203
		26	220	331	192	289	206	310	174	261	141	212	126	189
		28	196	295	173	260	186	279	158	238	130	196	117	176
		30	173	261	155	233	166	249	143	215	120	180	108	162
		32	152	229	137	206	147	221	128	193	109	164	99.3	149
		34	135	203	122	183	130	196	114	172	98.8	149	90.7	136
		36	120	181	108	163	116	175	102	153	88.9	134	82.4	124
		40	97.6	147	87.8	132	94.0	141	82.6	124	72.0	108	67.1	101
	Y-Y Axis	0	450	676	369	554	395	594	312	469	234	352	200	300
		10	366	551	299	449	275	414	222	334	171	257	147	220
		12	356	534	292	439	243	365	199	299	156	234	135	203
		14	340	510	282	423	209	314	174	261	139	209	122	183
		16	320	481	267	402	175	263	148	223	121	182	107	161
		18	297	447	251	377	143	214	124	186	104	156	92.8	140
		20	273	411	233	350	117	175	101	152	86.9	131	78.9	119
		22	249	374	214	321	97.1	146	84.5	127	72.6	109	66.1	99.3
		24	224	336	194	292	82.0	123	71.5	107	61.5	92.4	56.1	84.3
		26	200	300	175	264	70.1	105	61.2	92.0	52.7	79.3	48.1	72.3
		28	176	265	157	236	60.7	91.2	53.0	79.6	45.7	68.7	41.7	62.7
		30	155	232	139	209	53.0	79.6	46.3	69.6	39.9	60.0	36.5	54.9
		32	136	205	123	184								
		34	121	182	109	164								
		36	108	163	97.5	147								
		40	88.1	132	79.3	119								

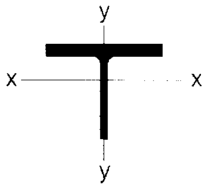
Properties

A_g (in.^{2})	16.3	14.9	13.7	12.2	10.7	10.00
r_x (in.)	3.03	3.01	3.25	3.22	3.21	3.20
r_y (in.)	2.90	2.89	1.84	1.83	1.81	1.80

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



WT10.5

Table 4-7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

F_y = 50 ksi

Shape			WT10.5×											
Wt/ft			31^c		27.5^c		24^c		28.5^c		25^c		22^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	159	239	126	190	97.5	147	148	223	117	176	89.2	134
		10	150	225	120	180	93.2	140	140	210	111	167	85.4	128
		12	146	219	117	176	91.3	137	136	205	109	164	83.7	126
		14	141	212	114	171	89.2	134	132	199	106	159	81.8	123
		16	136	205	110	166	86.7	130	128	192	103	155	79.7	120
		18	131	197	106	160	84.1	126	123	185	99.3	149	77.3	116
		20	125	188	102	154	81.2	122	118	177	95.5	144	74.8	112
		22	119	179	97.8	147	78.1	117	112	169	91.4	137	72.1	108
		24	113	169	93.2	140	74.9	113	106	160	87.2	131	69.2	104
		26	106	160	88.4	133	71.6	108	100	151	82.9	125	66.2	99.5
		28	99.6	150	83.6	126	68.1	102	94.3	142	78.4	118	63.1	94.9
		30	92.9	140	78.6	118	64.6	97.1	88.2	133	73.9	111	60.0	90.1
		32	86.3	130	73.7	111	61.0	91.7	82.1	123	69.3	104	56.8	85.3
		34	79.8	120	68.7	103	57.5	86.4	76.1	114	64.7	97.3	53.6	80.5
		36	73.4	110	63.8	96.0	53.9	81.0	70.2	106	60.3	90.6	50.4	75.7
		40	61.2	92.0	54.4	81.8	46.9	70.5	58.9	88.5	51.5	77.5	44.0	66.2
	Y-Y Axis	0	159	239	126	190	97.5	147	148	223	117	176	89.2	134
		10	117	176	90.5	136	66.4	99.9	95.9	144	73.3	110	55.2	82.9
		12	109	164	85.2	128	63.0	94.7	83.3	125	64.3	96.6	49.2	73.9
		14	99.9	150	78.7	118	58.6	88.1	70.2	105	54.6	82.1	42.5	63.9
		16	89.5	135	71.2	107	53.5	80.4	57.4	86.2	45.0	67.6	35.7	53.7
		18	78.8	118	63.3	95.1	47.9	72.1	46.2	69.4	36.6	54.9	29.4	44.1
		20	68.2	102	55.3	83.2	42.2	63.5	37.9	57.0	30.2	45.4	24.4	36.7
		22	57.9	87.0	47.5	71.4	36.5	54.9	31.6	47.6				
		24	49.2	74.0	40.6	61.0	31.4	47.3						
		26	42.3	63.6	35.0	52.7	27.3	41.0						
		28	36.8	55.2	30.5	45.8								

Properties

A_g (in.^{2})	9.13	8.10	7.07	8.37	7.36	6.49
r_x (in.)	3.21	3.23	3.26	3.29	3.30	3.31
r_y (in.)	1.77	1.73	1.66	1.35	1.30	1.26

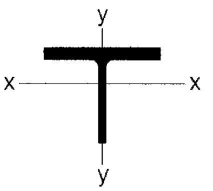
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



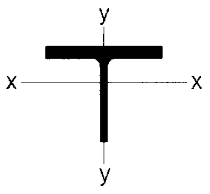
WT9

		Shape	WT9×											
		Wt/ft	87.5		79		71.5		65		59.5		53	
		Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	**X-X Axis**	**0**	768	1150	693	1040	630	946	572	860	525	789	466	701
		10	662	995	595	895	539	809	488	734	449	675	398	599
		12	620	932	557	837	503	756	455	684	420	631	372	559
		14	574	862	514	773	464	697	419	630	387	582	343	515
		16	525	789	469	705	422	634	381	573	352	530	312	468
		18	474	713	423	636	380	570	342	515	317	477	280	421
		20	423	636	377	566	337	507	304	456	282	424	249	374
		22	374	561	332	498	296	444	266	399	247	372	218	327
		24	326	490	288	433	256	385	230	345	214	322	188	283
		26	280	421	247	371	219	329	196	295	183	275	161	242
		28	241	363	213	320	189	284	169	254	158	238	139	209
		30	210	316	185	279	164	247	147	221	138	207	121	182
		32	185	278	163	245	144	217	129	195	121	182	106	160
		34	164	246	144	217	128	192	115	172	107	161	94.2	142
		36	146	220	129	194	114	172	102	154	95.6	144	84.0	126
		40	118	178	104	157	92.5	139	82.9	125	77.4	116	68.0	102
	Y-Y Axis	**0**	768	1150	693	1040	630	946	572	860	525	789	466	701
		10	660	992	593	891	536	805	483	727	438	659	384	577
		12	621	934	558	838	504	757	454	683	412	619	361	543
		14	579	870	519	780	469	704	422	635	383	576	335	504
		16	533	802	478	718	431	648	388	583	352	529	308	463
		18	486	731	435	653	392	589	352	530	320	480	279	420
		20	438	658	391	588	352	529	316	475	287	431	250	376
		22	391	587	348	523	313	470	281	422	254	382	222	333
		24	344	517	306	460	275	413	246	370	223	335	194	292
		26	300	451	266	400	238	358	213	320	193	290	167	252
		28	259	389	230	345	206	309	184	276	167	250	145	217
		30	226	339	200	301	179	270	160	241	145	218	126	190
		32	199	298	176	265	158	237	141	212	128	192	111	167
		34	176	264	156	235	140	210	125	188	113	170	98.6	148
		36	157	236	139	209	125	188	112	168	101	152	88.0	132
		40	127	191	113	170	101	152	90.5	136	82.1	123	71.4	107

Properties

	87.5	79	71.5	65	59.5	53
A_g (in.^{2})	25.7	23.2	21.0	19.1	17.5	15.6
r_x (in.)	2.66	2.63	2.60	2.58	2.60	2.59
r_y (in.)	2.76	2.74	2.72	2.70	2.69	2.66

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$



WT9

Table 4–7 (continued)

Available Strength in Axial Compression, kips

WT Shapes

F_y = 50 ksi

Shape			WT9×											
Wt/ft			**48.5**		**43[c]**		**38[c]**		**35.5[c]**		**32.5[c]**		**30[c]**	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to indicated axis	**X-X Axis**	**0**	427	642	356	535	275	414	301	453	251	377	210	316
		10	364	547	306	460	241	362	263	395	221	333	188	282
		12	339	510	286	430	227	341	248	372	210	315	178	268
		14	312	469	264	397	211	318	231	347	196	295	168	253
		16	284	426	241	363	195	293	213	320	182	274	157	236
		18	254	382	218	327	178	267	194	292	167	252	145	218
		20	225	338	194	291	161	241	175	263	152	229	133	200
		22	197	296	171	257	143	215	156	235	137	206	121	182
		24	170	255	148	223	127	190	138	208	122	184	109	164
		26	145	218	127	191	111	166	121	181	108	162	97.1	146
		28	125	188	110	165	95.6	144	104	157	94.1	142	85.8	129
		30	109	164	95.6	144	83.3	125	90.7	136	82.0	123	75.0	113
		32	95.6	144	84.0	126	73.2	110	79.8	120	72.1	108	65.9	99.1
		34	84.7	127	74.4	112	64.8	97.4	70.6	106	63.8	96.0	58.4	87.8
		36	75.5	114	66.4	99.8	57.8	86.9	63.0	94.7	57.0	85.6	52.1	78.3
		40	61.2	92.0	53.8	80.8	46.8	70.4	51.0	76.7	46.1	69.3	42.2	63.4
	Y-Y Axis	**0**	427	642	356	535	275	414	301	453	251	377	210	316
		10	349	525	287	431	221	331	201	302	172	258	147	221
		12	328	494	275	413	214	321	173	260	150	226	130	196
		14	305	459	258	387	203	305	145	218	127	192	112	169
		16	280	421	238	358	190	286	117	176	105	158	94.2	142
		18	254	382	217	326	175	263	93.7	141	84.8	127	77.0	116
		20	228	343	195	293	160	240	76.4	115	69.2	104	63.0	94.6
		22	202	303	173	260	144	216	63.5	95.4	57.6	86.5	52.4	78.7
		24	177	265	152	228	128	193	53.6	80.5	48.6	73.0	44.2	66.5
		26	152	229	132	198	113	170	45.8	68.8	41.5	62.4	37.8	56.9
		28	132	198	114	171	98.4	148	39.6	59.5	35.9	54.0	32.7	49.2
		30	115	173	99.5	150	86.1	129						
		32	101	152	87.7	132	75.9	114						
		34	89.8	135	77.9	117	67.5	101						
		36	80.2	120	69.6	105	60.3	90.7						
		40	65.1	97.8	56.5	85.0	49.0	73.7						

Properties

	48.5	43[c]	38[c]	35.5[c]	32.5[c]	30[c]
A_g (in.^{2})	14.3	12.7	11.2	10.4	9.55	8.82
r_x (in.)	2.56	2.55	2.54	2.74	2.72	2.71
r_y (in.)	2.65	2.63	2.61	1.70	1.69	1.68

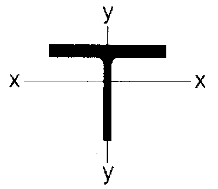
ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

F_y = 50 ksi



WT9

Shape		WT9×									
Wt/ft		27.5^c		25^c		23^c		20^c		17.5^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis — X-X Axis	0	178	268	137	206	129	194	87.2	131	70.8	106
	10	160	241	125	188	118	178	81.5	122	66.6	100
	12	153	230	120	181	114	171	79.1	119	64.8	97.3
	14	145	218	115	172	109	163	76.3	115	62.7	94.3
	16	136	204	109	163	103	155	73.2	110	60.4	90.8
	18	127	190	102	154	97.2	146	69.9	105	57.9	87.0
	20	117	176	95.5	144	91.0	137	66.4	99.7	55.2	83.0
	22	107	161	88.5	133	84.6	127	62.7	94.2	52.4	78.8
	24	97.1	146	81.5	123	78.1	117	58.8	88.4	49.5	74.4
	26	87.4	131	74.5	112	71.6	108	54.9	82.6	46.5	69.9
	28	78.1	117	67.6	102	65.1	97.9	51.0	76.7	43.5	65.4
	30	69.0	104	60.9	91.6	58.9	88.5	47.1	70.8	40.5	60.9
	32	60.7	91.2	54.5	81.9	52.9	79.4	43.3	65.1	37.5	56.3
	34	53.8	80.8	48.3	72.7	47.0	70.7	39.6	59.5	34.5	51.9
	36	47.9	72.1	43.1	64.8	41.9	63.0	35.9	54.0	31.7	47.6
	40	38.8	58.4	34.9	52.5	34.0	51.1	29.3	44.0	26.2	39.3
Y-Y Axis	0	178	268	137	206	129	194	87.2	131	70.8	106
	10	125	188	99.1	149	80.1	120	58.4	87.8	45.0	67.6
	12	112	168	90.3	136	67.5	101	51.1	76.8	39.5	59.4
	14	97.2	146	80.2	121	54.9	82.6	43.4	65.3	33.7	50.6
	16	82.5	124	69.8	105	43.3	65.1	35.9	54.0	27.9	41.9
	18	68.4	103	59.4	89.3	34.7	52.1	29.0	43.6	22.6	34.0
	20	56.0	84.2	49.5	74.4	28.3	42.6	23.8	35.7	18.7	28.1
	22	46.7	70.2	41.3	62.1						
	24	39.5	59.3	35.0	52.6						
	26	33.8	50.8	30.0	45.1						

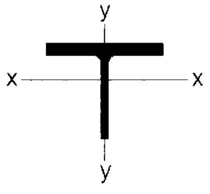
Properties

	27.5	25	23	20	17.5
A_g (in.^{2})	8.10	7.33	6.77	5.88	5.15
r_x (in.)	2.71	2.70	2.77	2.76	2.79
r_y (in.)	1.67	1.65	1.29	1.27	1.22

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



WT8

Table 4-7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

$F_y = 50$ ksi

Shape			WT8×											
Wt/ft			50		44.5		38.5^c		33.5^c		28.5^c		25^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	441	663	392	588	335	504	254	382	237	355	182	274
		10	360	541	319	479	273	410	211	318	199	300	157	236
		12	330	495	292	438	249	374	195	293	185	278	147	220
		14	297	446	262	394	223	336	177	266	169	254	136	204
		16	263	395	232	348	197	297	159	239	153	230	124	186
		18	229	344	202	303	171	258	140	210	136	204	112	168
		20	197	295	173	259	146	220	122	183	120	180	99.5	150
		22	166	249	145	218	123	185	104	157	104	156	87.6	132
		24	139	209	122	183	103	155	88.0	132	88.3	133	76.2	115
		26	119	178	104	156	87.9	132	75.0	113	75.2	113	65.3	98.2
		28	102	154	89.5	135	75.8	114	64.7	97.2	64.9	97.5	56.3	84.7
		30	89.0	134	78.0	117	66.0	99.3	56.3	84.7	56.5	85.0	49.1	73.8
		32	78.2	118	68.5	103	58.0	87.2	49.5	74.4	49.7	74.7	43.1	64.8
		34	69.3	104	60.7	91.2	51.4	77.3	43.9	65.9	44.0	66.1	38.2	57.4
		36	61.8	92.9	54.1	81.4	45.9	68.9	39.1	58.8	39.3	59.0	34.1	51.2
		40									31.8	47.8	27.6	41.5
	Y-Y Axis	0	441	663	392	588	335	504	254	382	237	355	182	274
		10	363	546	319	479	270	406	205	308	154	231	122	184
		12	338	509	297	446	253	381	196	294	130	196	106	159
		14	311	467	273	410	233	351	183	274	107	161	88.7	133
		16	282	424	247	372	211	317	168	252	84.7	127	72.2	109
		18	253	380	221	332	188	283	152	228	67.5	101	57.7	86.8
		20	223	335	195	293	166	249	136	204	55.0	82.6	47.1	70.8
		22	194	292	170	255	144	216	120	180	45.6	68.6	39.2	58.8
		24	167	251	145	218	123	184	104	157	38.5	57.8	33.0	49.6
		26	142	214	124	186	105	158	89.7	135	32.8	49.4	28.2	42.4
		28	123	185	107	161	90.7	136	77.7	117				
		30	107	161	93.5	140	79.2	119	67.8	102				
		32	94.4	142	82.2	124	69.7	105	59.7	89.8				
		34	83.6	126	72.9	110	61.8	92.9	53.0	79.7				
		36	74.6	112	65.1	97.8	55.2	83.0	47.4	71.2				
		40	60.5	91.0	52.8	79.3	44.8	67.3	38.4	57.8				

Properties

	50	44.5	38.5^c	33.5^c	28.5^c	25^c
A_g (in.^{2})	14.7	13.1	11.3	9.84	8.39	7.37
r_x (in.)	2.28	2.27	2.24	2.22	2.41	2.40
r_y (in.)	2.51	2.49	2.47	2.46	1.60	1.59

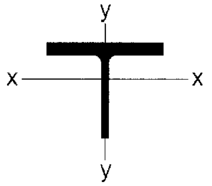
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with $F_y = 50$ ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT8

Shape		WT8×									
Wt/ft		22.5^c		20^c		18^c		15.5^c		13^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis — X-X Axis	0	144	217	102	154	87.6	132	65.6	98.5	46.7	70.2
	10	126	190	91.8	138	79.2	119	60.3	90.6	43.6	65.5
	12	119	179	87.6	132	75.8	114	58.1	87.3	42.3	63.5
	14	111	167	82.8	124	71.9	108	55.6	83.6	40.7	61.2
	16	102	154	77.5	117	67.7	102	52.9	79.5	39.1	58.7
	18	93.4	140	72.0	108	63.2	95.0	49.9	75.1	37.3	56.0
	20	84.3	127	66.3	99.7	58.5	88.0	46.8	70.4	35.3	53.1
	22	75.4	113	60.5	91.0	53.8	80.9	43.7	65.6	33.3	50.1
	24	66.6	100	54.8	82.3	49.0	73.7	40.4	60.7	31.2	47.0
	26	58.3	87.6	49.1	73.8	44.3	66.6	37.2	55.8	29.1	43.8
	28	50.4	75.7	43.7	65.7	39.8	59.8	33.9	51.0	27.0	40.6
	30	43.9	65.9	38.4	57.8	35.4	53.2	30.8	46.3	24.9	37.4
	32	38.6	57.9	33.8	50.8	31.2	46.8	27.7	41.7	22.8	34.3
	34	34.2	51.3	29.9	45.0	27.6	41.5	24.8	37.3	20.8	31.3
	36	30.5	45.8	26.7	40.1	24.6	37.0	22.1	33.2	18.9	28.4
	40					19.9	30.0	17.9	26.9	15.3	23.1
Y-Y Axis	0	144	217	102	154	87.6	132	65.6	98.5	46.7	70.2
	10	99.4	149	74.6	112	61.4	92.3	41.7	62.6	29.2	43.9
	12	87.5	132	67.5	101	55.8	83.9	35.7	53.6	25.5	38.3
	14	74.8	112	59.6	89.6	49.5	74.3	29.5	44.4	21.5	32.3
	16	62.4	93.7	51.5	77.4	42.8	64.3	23.6	35.5	17.6	26.4
	18	50.6	76.0	43.5	65.4	36.2	54.4	19.0	28.5	14.2	21.4
	20	41.3	62.1	36.0	54.1	30.0	45.0				
	22	34.4	51.7	30.0	45.1	25.1	37.7				
	24	29.0	43.7	25.4	38.1	21.2	31.9				
	26	24.8	37.3	21.7	32.7						

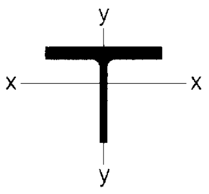
Properties

	22.5	20	18	15.5	13
A_g (in.^{2})	6.63	5.89	5.29	4.56	3.84
r_x (in.)	2.39	2.37	2.41	2.45	2.47
r_y (in.)	1.57	1.56	1.52	1.17	1.12

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



WT7

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

$F_y = 50$ ksi

Shape		WT7×											
Wt/ft		66		60		54.5		49.5		45		41	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis — X-X Axis	0	580	872	529	795	480	721	436	656	396	596	360	541
	10	408	613	369	555	330	497	300	450	270	406	265	398
	12	349	525	315	473	281	422	254	382	229	343	231	348
	14	291	437	261	393	231	347	209	314	187	282	197	296
	16	235	353	210	316	185	277	167	250	149	223	164	246
	18	186	280	167	250	146	219	132	198	117	176	133	199
	20	151	227	135	203	118	178	107	160	95.1	143	107	161
	22	125	187	111	168	97.6	147	88.1	132	78.6	118	88.8	133
	24	105	158	93.7	141	82.0	123	74.1	111	66.1	99.3	74.6	112
	26	89.3	134	79.8	120	69.9	105	63.1	94.8	56.3	84.6	63.6	95.6
	28	77.0	116	68.8	103	60.3	90.6					54.8	82.4
	30											47.8	71.8
Y-Y Axis	0	580	872	529	795	480	721	436	656	396	596	360	541
	10	533	802	484	728	438	659	396	595	358	538	298	447
	12	516	776	469	705	424	638	383	576	346	520	276	416
	14	497	747	451	678	408	613	369	555	333	501	253	381
	16	475	715	432	649	390	587	353	530	319	479	229	344
	18	452	680	410	617	371	558	335	504	303	455	204	307
	20	428	643	388	583	351	527	317	476	286	430	180	270
	22	402	604	364	548	329	495	298	447	269	404	156	235
	24	376	564	340	511	308	462	278	417	251	377	133	200
	26	349	524	316	475	285	429	258	387	233	350	114	171
	28	322	484	291	438	263	396	237	357	214	322	98.2	148
	30	296	444	267	402	241	363	218	327	196	295	85.6	129
	32	270	405	244	366	220	331	198	298	179	268	75.3	113
	34	245	367	221	332	199	299	179	269	162	243	66.7	100
	36	220	331	198	298	179	269	161	242	145	218	59.5	89.5
	40	178	268	161	242	145	218	130	196	118	177	48.2	72.5

Properties

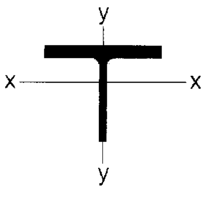
	66	60	54.5	49.5	45	41
A_g (in.^{2})	19.4	17.7	16.0	14.6	13.2	12.0
r_x (in.)	1.73	1.71	1.68	1.67	1.66	1.85
r_y (in.)	3.76	3.74	3.73	3.71	3.70	2.48

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT7

Shape			WT7×											
Wt/ft			37		34		30.5^c		26.5^c		24^c		21.5^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	326	490	299	450	261	392	223	336	187	281	147	220
		10	237	356	217	326	190	285	168	253	143	216	116	174
		12	206	310	188	283	165	248	148	223	128	192	104	157
		14	175	262	159	239	140	210	128	192	111	167	92.5	139
		16	144	217	131	197	116	174	108	162	95.0	143	80.3	121
		18	116	174	105	158	93.2	140	88.8	133	79.4	119	68.5	103
		20	93.9	141	85.2	128	75.5	113	72.0	108	64.9	97.5	57.3	86.1
		22	77.6	117	70.4	106	62.4	93.7	59.5	89.4	53.6	80.6	47.3	71.1
		24	65.2	98.0	59.2	88.9	52.4	78.8	50.0	75.1	45.1	67.7	39.8	59.8
		26	55.6	83.5	50.4	75.8	44.7	67.1	42.6	64.0	38.4	57.7	33.9	50.9
		28	47.9	72.0	43.5	65.3	38.5	57.9	36.7	55.2	33.1	49.8	29.2	43.9
		30	41.7	62.7	37.9	56.9	33.5	50.4	32.0	48.1	28.8	43.3	25.4	38.2
	Y-Y Axis	0	326	490	299	450	261	392	223	336	187	281	147	220
		10	269	404	245	368	212	318	166	250	140	211	112	169
		12	250	375	227	342	199	299	148	222	126	189	102	154
		14	229	344	208	313	183	275	128	193	111	166	91.3	137
		16	207	311	188	283	165	249	109	164	95.0	143	79.8	120
		18	184	277	168	252	147	222	90.8	136	79.9	120	68.4	103
		20	162	244	147	221	130	195	74.1	111	65.8	98.9	57.6	86.5
		22	141	211	128	192	112	169	61.4	92.3	54.6	82.0	47.8	71.8
		24	120	180	109	164	95.9	144	51.7	77.7	46.0	69.1	40.3	60.6
		26	102	154	92.9	140	81.9	123	44.1	66.3	39.3	59.0	34.4	51.7
		28	88.5	133	80.2	121	70.8	106	38.1	57.3	33.9	50.9	29.7	44.7
		30	77.1	116	69.9	105	61.7	92.8	33.2	49.9	29.6	44.4	25.9	39.0
		32	67.8	102	61.5	92.4	54.3	81.6	29.2	43.9				
		34	60.1	90.4	54.5	81.9	48.2	72.4						
		36	53.6	80.6	48.7	73.1	43.0	64.6						
		40	43.5	65.4	39.4	59.3	34.9	52.4						

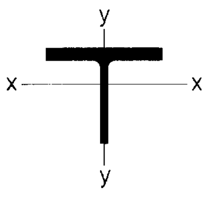
Properties

	37	34	30.5^c	26.5^c	24^c	21.5^c
A_g (in.^{2})	10.9	9.99	8.96	7.80	7.07	6.31
r_x (in.)	1.82	1.81	1.80	1.88	1.88	1.86
r_y (in.)	2.48	2.46	2.45	1.92	1.91	1.89

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



WT7

Table 4–7 (continued)

Available Strength in Axial Compression, kips

WT Shapes

F_y = 50 ksi

Shape			WT7×									
Wt/ft			19^c		17^c		15^c		13^c		11^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	127	190	100	150	80.7	121	61.9	93.1	43.6	65.5
		10	105	157	84.5	127	69.5	104	54.6	82.1	39.3	59.1
		12	96.2	145	78.5	118	65.0	97.7	51.7	77.6	37.6	56.5
		14	87.1	131	71.9	108	60.2	90.4	48.4	72.7	35.6	53.5
		16	77.6	117	65.0	97.7	55.0	82.7	44.9	67.5	33.5	50.3
		18	68.1	102	58.0	87.2	49.7	74.7	41.2	61.9	31.2	46.9
		20	58.9	88.5	51.0	76.7	44.3	66.6	37.5	56.3	28.8	43.3
		22	50.2	75.4	44.3	66.6	39.1	58.8	33.7	50.7	26.4	39.7
		24	42.2	63.4	37.9	56.9	34.1	51.2	30.0	45.1	24.0	36.1
		26	35.9	54.0	32.3	48.5	29.3	44.0	26.5	39.8	21.7	32.6
		28	31.0	46.6	27.8	41.8	25.2	37.9	23.1	34.7	19.4	29.1
		30	27.0	40.6	24.2	36.4	22.0	33.1	20.1	30.2	17.2	25.9
		32	23.7	35.7	21.3	32.0	19.3	29.1	17.7	26.5	15.1	22.7
		34	21.0	31.6	18.9	28.4	17.1	25.7	15.6	23.5	13.4	20.1
	Y-Y Axis	0	127	190	100	150	80.7	121	61.9	93.1	43.6	65.5
		10	86.3	130	69.5	104	55.0	82.7	35.6	53.5	25.6	38.5
		12	75.0	113	61.4	92.3	49.2	73.9	28.9	43.4	21.3	32.0
		14	63.2	95.0	52.7	79.3	42.6	64.1	22.4	33.7	17.1	25.6
		16	51.8	77.9	44.1	66.3	36.0	54.1	17.4	26.1	13.4	20.1
		18	41.5	62.4	35.9	54.0	29.6	44.5	13.9	20.9		
		20	33.9	50.9	29.4	44.1	24.3	36.5				
		22	28.1	42.3	24.4	36.7	20.3	30.5				
		24	23.7	35.7	20.6	31.0	17.2	25.8				

Properties

	19	17	15	13	11
A_g (in.^{2})	5.58	5.00	4.42	3.85	3.25
r_x (in.)	2.04	2.04	2.07	2.12	2.14
r_y (in.)	1.55	1.53	1.49	1.08	1.04

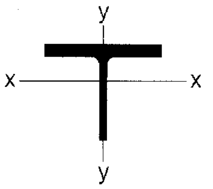
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT6

Shape			WT6×											
Wt/ft			29		26.5		25		22.5		20^c		17.5^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	255	384	233	350	218	328	196	295	155	233	132	199
		4	237	356	216	325	205	307	183	276	146	219	126	190
		6	215	324	197	297	188	283	169	254	135	203	119	179
		8	189	284	173	261	168	252	150	226	122	183	110	165
		10	159	240	147	221	145	218	129	194	106	159	98.8	149
		12	130	195	120	180	121	182	108	162	89.8	135	86.9	131
		14	101	152	94.2	142	97.6	147	86.8	130	73.8	111	74.7	112
		16	77.7	117	72.3	109	76.2	115	67.6	102	58.8	88.4	62.7	94.3
		18	61.4	92.3	57.1	85.9	60.2	90.5	53.4	80.3	46.5	69.8	51.4	77.3
		20	49.7	74.8	46.3	69.5	48.8	73.3	43.3	65.1	37.6	56.6	41.7	62.6
		22	41.1	61.8	38.2	57.5	40.3	60.6	35.8	53.8	31.1	46.7	34.4	51.7
		24	34.5	51.9	32.1	48.3	33.9	50.9	30.1	45.2	26.1	39.3	28.9	43.5
		26					28.8	43.4	25.6	38.5	22.3	33.5	24.6	37.0
		28											21.3	31.9
	Y-Y Axis	0	255	384	233	350	218	328	196	295	155	233	132	199
		4	242	364	219	329	202	304	170	255	133	200	113	169
		6	235	353	212	318	192	288	167	251	131	197	108	163
		8	224	337	202	304	178	268	159	239	126	190	99.4	149
		10	211	318	191	287	162	244	145	218	117	177	87.4	131
		12	197	296	177	267	144	217	129	194	106	159	74.2	112
		14	181	272	163	245	126	189	112	168	93.5	140	61.0	91.8
		16	164	246	147	221	107	161	95.1	143	80.7	121	48.6	73.0
		18	146	220	132	198	89.1	134	78.9	119	68.3	103	38.6	58.0
		20	129	194	116	174	72.8	109	64.2	96.6	56.5	85.0	31.4	47.2
		22	112	169	101	151	60.2	90.5	53.2	80.0	46.9	70.4	26.0	39.1
		24	96.2	145	85.9	129	50.7	76.2	44.8	67.4	39.5	59.3	21.9	33.0
		26	82.1	123	73.3	110	43.2	65.0	38.3	57.5	33.7	50.6		
		28	70.9	107	63.3	95.1	37.3	56.1	33.0	49.6	29.1	43.7		
		30	61.8	92.9	55.2	82.9	32.5	48.9	28.8	43.3	25.4	38.1		
		32	54.3	81.7	48.5	72.9	28.6	43.0	25.3	38.1	22.3	33.5		

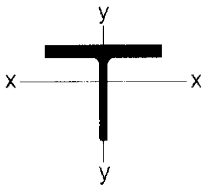
Properties

A_g (in.^{2})	8.52	7.78	7.30	6.56	5.84	5.17
r_x (in.)	1.50	1.51	1.60	1.59	1.57	1.76
r_y (in.)	2.51	2.48	1.96	1.95	1.94	1.54

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



WT6

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

F_y = 50 ksi

Shape			WT6×											
Wt/ft			15^c		13^c		11^c		9.5^c		8^c		7^c	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to indicated axis	**X-X Axis**	**0**	**93.2**	140	**64.9**	97.5	**69.0**	104	**49.9**	74.9	**38.0**	57.1	**28.1**	42.2
		4	**89.7**	135	**62.9**	94.5	**66.8**	100	**48.5**	72.9	**37.1**	55.7	**27.5**	41.3
		6	**85.4**	128	**60.5**	90.9	**64.0**	96.3	**46.8**	70.4	**36.0**	54.1	**26.8**	40.3
		8	**79.8**	120	**57.2**	86.0	**60.4**	90.8	**44.6**	67.1	**34.4**	51.8	**25.9**	38.9
		10	**73.1**	110	**53.4**	80.2	**56.1**	84.3	**41.9**	63.0	**32.6**	49.0	**24.7**	37.1
		12	**65.7**	98.7	**49.0**	73.6	**51.2**	76.9	**38.8**	58.4	**30.5**	45.8	**23.3**	35.1
		14	**57.9**	87.0	**44.2**	66.5	**46.0**	69.1	**35.5**	53.3	**28.1**	42.3	**21.8**	32.8
		16	**50.0**	75.2	**39.3**	59.1	**40.6**	61.0	**32.0**	48.1	**25.6**	38.5	**20.2**	30.4
		18	**42.4**	63.7	**34.4**	51.8	**35.2**	53.0	**28.4**	42.7	**23.1**	34.7	**18.5**	27.8
		20	**35.2**	52.9	**29.7**	44.6	**30.1**	45.2	**24.9**	37.4	**20.6**	30.9	**16.8**	25.2
		22	**29.1**	43.7	**25.2**	37.8	**25.2**	37.9	**21.5**	32.4	**18.1**	27.2	**15.1**	22.6
		24	**24.4**	36.7	**21.2**	31.8	**21.2**	31.9	**18.3**	27.5	**15.7**	23.6	**13.4**	20.1
		26	**20.8**	31.3	**18.0**	27.1	**18.1**	27.2	**15.6**	23.5	**13.4**	20.2	**11.8**	17.7
		28	**17.9**	27.0	**15.5**	23.4	**15.6**	23.4	**13.5**	20.2	**11.6**	17.4	**10.2**	15.4
		30					**13.6**	20.4	**11.7**	17.6	**10.1**	15.2	**8.90**	13.4
		32									**8.87**	13.3	**7.82**	11.8
	Y-Y Axis	**0**	**93.2**	140	**64.9**	97.5	**69.0**	104	**49.9**	74.9	**38.0**	57.1	**28.1**	42.2
		4	**78.1**	117	**54.0**	81.2	**52.3**	78.6	**37.0**	55.6	**25.6**	38.5	**18.6**	27.9
		6	**76.1**	114	**53.1**	79.8	**43.7**	65.6	**31.9**	47.9	**22.3**	33.5	**16.5**	24.9
		8	**71.6**	108	**50.9**	76.5	**33.0**	49.5	**25.0**	37.5	**17.6**	26.5	**13.6**	20.4
		10	**64.6**	97.1	**47.2**	70.9	**22.8**	34.3	**17.9**	27.0	**12.7**	19.1	**10.2**	15.3
		12	**56.4**	84.8	**42.4**	63.8	**16.2**	24.3	**12.8**	19.3	**9.27**	13.9	**7.53**	11.3
		14	**47.8**	71.9	**37.2**	55.9	**12.0**	18.1						
		16	**39.5**	59.4	**31.8**	47.8								
		18	**31.8**	47.8	**26.7**	40.1								
		20	**25.9**	38.9	**21.9**	32.9								
		22	**21.5**	32.3	**18.2**	27.4								
		24	**18.1**	27.3	**15.4**	23.1								

Properties

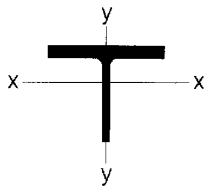
	15	13	11	9.5	8	7
A_g (in.^{2})	4.40	3.82	3.24	2.79	2.36	2.08
r_x (in.)	1.75	1.75	1.90	1.90	1.92	1.92
r_y (in.)	1.52	1.51	0.847	0.821	0.773	0.753

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT5

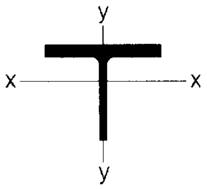
Shape		WT5×									
Wt/ft		22.5		19.5		16.5		15		13^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to indicated axis — X-X Axis	0	199	298	172	258	145	218	132	199	103	155
	4	178	268	154	231	131	196	122	184	95.7	144
	6	155	233	134	202	114	172	110	166	87.2	131
	8	128	193	111	167	95.1	143	96.0	144	76.7	115
	10	100	151	86.7	130	74.9	113	80.1	120	65.0	97.6
	12	74.2	112	64.1	96.3	55.9	84.0	64.3	96.6	53.0	79.7
	14	54.5	81.9	47.1	70.8	41.1	61.7	49.4	74.3	41.7	62.7
	16	41.7	62.7	36.1	54.2	31.4	47.3	37.8	56.9	32.0	48.2
	18	33.0	49.6	28.5	42.8	24.8	37.3	29.9	44.9	25.3	38.1
	20	26.7	40.1	23.1	34.7	20.1	30.2	24.2	36.4	20.5	30.8
	22							20.0	30.1	17.0	25.5
	24							16.8	25.3	14.2	21.4
Y-Y Axis	0	199	298	172	258	145	218	132	199	103	155
	4	187	281	160	241	133	199	115	173	87.0	131
	6	178	267	152	229	126	189	103	156	81.5	123
	8	166	249	142	213	117	176	89.2	134	71.6	108
	10	151	227	129	194	106	160	73.6	111	59.8	89.9
	12	135	202	115	173	94.5	142	58.0	87.1	47.9	71.9
	14	118	177	100	150	82.1	123	43.7	65.7	36.6	55.0
	16	101	151	85.4	128	69.8	105	33.6	50.5	28.2	42.4
	18	84.5	127	71.3	107	57.9	87.0	26.6	40.0	22.4	33.7
	20	69.2	104	58.3	87.6	47.2	70.9	21.6	32.5	18.2	27.3
	22	57.3	86.1	48.2	72.5	39.1	58.7	17.9	26.9	15.1	22.6
	24	48.2	72.4	40.6	61.0	32.9	49.4				
	26	41.1	61.7	34.6	52.0	28.1	42.2				
	28	35.4	53.2	29.9	44.9	24.2	36.4				
	30	30.9	46.4	26.0	39.1	21.1	31.7				
	32	27.1	40.8	22.9	34.4	18.6	27.9				

Properties

	22.5	19.5	16.5	15	13^c
A_g (in.^{2})	6.63	5.73	4.85	4.42	3.81
r_x (in.)	1.24	1.24	1.26	1.45	1.44
r_y (in.)	2.01	1.98	1.94	1.37	1.36

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 50 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.



WT5

Table 4–7 (continued)
Available Strength in Axial Compression, kips
WT Shapes

$F_y = 50$ ksi

Shape			WT5×									
Wt/ft			11[c]		9.5[c]		8.5[c]		7.5[c]		6	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	81.2	122	73.4	110	63.0	94.7	53.5	80.4	31.4	47.2
		4	76.0	114	69.0	104	59.4	89.3	50.6	76.1	30.1	45.3
		6	70.0	105	63.8	95.9	55.2	83.0	47.2	71.0	28.6	43.1
		8	62.3	93.6	57.3	86.1	49.9	74.9	42.9	64.5	26.7	40.1
		10	53.6	80.6	49.9	74.9	43.7	65.7	37.9	56.9	24.3	36.6
		12	44.7	67.2	42.1	63.2	37.2	55.9	32.5	48.9	21.8	32.7
		14	36.0	54.1	34.4	51.7	30.8	46.2	27.2	40.9	19.1	28.7
		16	28.1	42.2	27.3	41.0	24.7	37.1	22.1	33.2	16.4	24.6
		18	22.2	33.3	21.5	32.4	19.5	29.3	17.6	26.4	13.8	20.7
		20	18.0	27.0	17.4	26.2	15.8	23.8	14.2	21.4	11.3	17.1
		22	14.8	22.3	14.4	21.7	13.1	19.6	11.8	17.7	9.38	14.1
		24	12.5	18.8	12.1	18.2	11.0	16.5	9.87	14.8	7.88	11.8
		26					9.35	14.1	8.41	12.6	6.71	10.1
	Y-Y Axis	0	81.2	122	73.4	110	63.0	94.7	53.5	80.4	31.4	47.2
		4	65.3	98.2	55.7	83.6	45.3	68.1	35.7	53.7	20.8	31.3
		6	62.1	93.4	44.7	67.2	36.6	55.0	28.9	43.5	18.0	27.1
		8	55.6	83.5	32.2	48.4	26.2	39.3	20.5	30.8	14.0	21.1
		10	46.9	70.5	21.5	32.3	17.5	26.3	13.9	20.8	9.99	15.0
		12	37.9	56.9	15.1	22.7	12.4	18.6	9.90	14.9	7.25	10.9
		14	29.2	43.9	11.2	16.8	9.21	13.8				
		16	22.6	34.0								
		18	18.0	27.0								
		20	14.6	22.0								
		22	12.1	18.2								

Properties

A_g (in.^{2})	3.24	2.81	2.50	2.21	1.77
r_x (in.)	1.46	1.54	1.56	1.57	1.57
r_y (in.)	1.33	0.874	0.844	0.810	0.785

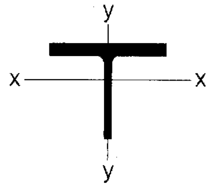
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT4

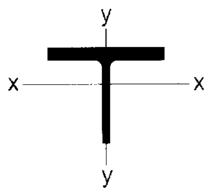
Shape			WT4×									
Wt/ft			33.5		29		24		20		17.5	
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	295	443	256	384	211	317	176	264	154	231
		4	253	380	218	328	177	267	148	222	129	193
		6	209	314	179	270	143	215	119	179	103	154
		8	160	241	136	205	106	159	88.1	132	75.0	113
		10	113	170	95.2	143	71.5	107	59.8	89.9	50.3	75.5
		12	78.8	118	66.1	99.4	49.7	74.6	41.5	62.4	34.9	52.5
		14	57.9	87.0	48.6	73.0	36.5	54.8	30.5	45.9	25.6	38.5
		16	44.3	66.6	37.2	55.9	27.9	42.0	23.4	35.1	19.6	29.5
	Y-Y Axis	0	295	443	256	384	211	317	176	264	154	231
		4	283	425	245	368	202	303	167	251	146	219
		6	270	405	233	351	192	289	159	238	138	208
		8	253	380	218	328	179	270	148	222	129	194
		10	232	349	200	301	165	247	135	203	118	177
		12	210	315	180	271	148	222	121	182	106	159
		14	186	279	159	240	130	196	107	160	92.7	139
		16	162	243	138	208	113	169	91.7	138	79.7	120
		18	138	207	117	176	95.6	144	77.3	116	67.1	101
		20	115	173	97.7	147	79.3	119	63.8	95.9	55.3	83.1
		22	95.4	143	80.8	121	65.6	98.6	52.8	79.3	45.7	68.7
		24	80.2	121	67.9	102	55.1	82.9	44.4	66.7	38.5	57.8
		26	68.3	103	57.9	87.0	47.0	70.6	37.8	56.8	32.8	49.3
		28	58.9	88.6	49.9	75.0	40.5	60.9	32.6	49.0	28.3	42.5
		30	51.3	77.2	43.5	65.4	35.3	53.1	28.4	42.7	24.6	37.0
		32	45.1	67.8	38.2	57.5	31.0	46.6	25.0	37.5	21.7	32.6

Properties

	33.5	29	24	20	17.5
A_g (in.^{2})	9.84	8.54	7.05	5.87	5.14
r_x (in.)	1.05	1.03	0.986	0.988	0.968
r_y (in.)	2.12	2.10	2.08	2.04	2.03

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



WT4

Table 4-7 (continued)

Available Strength in Axial Compression, kips

WT Shapes

F_y = 50 ksi

Shape		WT4×							
Wt/ft		15.5		14		12		10.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to indicated axis — **X-X Axis**	**0**	**137**	205	**123**	185	**106**	159	**92.3**	139
	4	**114**	171	**105**	157	**89.5**	134	**80.8**	121
	6	**91.2**	137	**85.3**	128	**72.5**	109	**68.4**	103
	8	**66.6**	100	**63.9**	96.1	**53.9**	81.1	**54.2**	81.4
	10	**44.7**	67.1	**44.1**	66.3	**36.9**	55.4	**40.1**	60.3
	12	**31.0**	46.6	**30.6**	46.0	**25.6**	38.5	**28.3**	42.5
	14	**22.8**	34.2	**22.5**	33.8	**18.8**	28.3	**20.8**	31.2
	16	**17.4**	26.2	**17.2**	25.9	**14.4**	21.6	**15.9**	23.9
	18							**12.6**	18.9
Y-Y Axis	**0**	**137**	205	**123**	185	**106**	159	**92.3**	139
	4	**128**	192	**113**	170	**96.4**	145	**79.3**	119
	6	**122**	183	**105**	158	**89.1**	134	**69.9**	105
	8	**114**	171	**93.9**	141	**79.8**	120	**58.5**	87.9
	10	**104**	156	**81.5**	122	**69.2**	104	**46.4**	69.7
	12	**92.7**	139	**68.5**	103	**58.0**	87.2	**34.8**	52.3
	14	**81.2**	122	**55.7**	83.8	**47.1**	70.8	**25.7**	38.6
	16	**69.7**	105	**43.8**	65.9	**36.9**	55.5	**19.7**	29.7
	18	**58.5**	87.9	**34.7**	52.1	**29.2**	44.0	**15.6**	23.5
	20	**48.1**	72.2	**28.1**	42.3	**23.7**	35.7	**12.7**	19.1
	22	**39.8**	59.8	**23.3**	35.0	**19.6**	29.5		
	24	**33.4**	50.3	**19.6**	29.4	**16.5**	24.8		
	26	**28.5**	42.9	**16.7**	25.1	**14.1**	21.1		
	28	**24.6**	37.0						
	30	**21.4**	32.2						
	32	**18.8**	28.3						

Properties

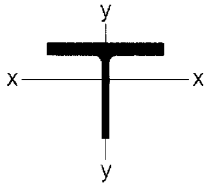
	15.5	14	12	10.5
A_g (in.^{2})	4.56	4.12	3.54	3.08
I (in.^{4})	0.969	1.01	0.999	1.12
r (in.)	2.02	1.62	1.61	1.26

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

$F_y = 50$ ksi

Table 4-7 (continued)
Available Strength in Axial Compression, kips
WT Shapes



WT4

Shape		WT4×							
Wt/ft		9		7.5		6.5		5^c	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to indicated axis									
X-X Axis	0	78.8	118	66.5	99.9	57.4	86.3	32.6	49.0
	4	69.2	104	59.3	89.1	51.4	77.2	29.9	45.0
	6	58.8	88.4	51.4	77.3	44.7	67.1	26.9	40.4
	8	46.8	70.4	42.1	63.3	36.7	55.2	23.2	34.8
	10	34.9	52.5	32.6	49.0	28.6	42.9	19.1	28.7
	12	24.7	37.1	23.8	35.7	20.9	31.5	15.1	22.7
	14	18.1	27.3	17.5	26.2	15.4	23.1	11.4	17.2
	16	13.9	20.9	13.4	20.1	11.8	17.7	8.75	13.2
	18	11.0	16.5	10.6	15.9	9.31	14.0	6.92	10.4
	20			8.55	12.9	7.54	11.3	5.60	8.42
Y-Y Axis	0	78.8	118	66.5	99.9	57.4	86.3	32.6	49.0
	4	65.2	98.0	48.1	72.3	38.5	57.8	23.0	34.6
	6	57.5	86.4	37.6	56.6	30.1	45.2	19.6	29.4
	8	48.0	72.2	26.3	39.6	20.7	31.2	14.7	22.1
	10	37.9	56.9	17.3	26.0	13.6	20.5	10.1	15.3
	12	28.1	42.3	12.1	18.2	9.61	14.4	7.23	10.9
	14	20.8	31.3	8.94	13.4	7.11	10.7	5.38	8.09
	16	16.0	24.1						
	18	12.7	19.1						
	20	10.3	15.5						

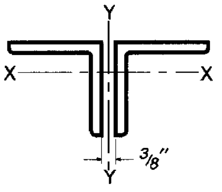
Properties

	9	7.5	6.5	5
A_g (in.^{2})	2.63	2.22	1.92	1.48
I (in.^{4})	1.14	1.22	1.23	1.20
r (in.)	1.23	0.876	0.843	0.840

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 50$ ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



2L8

Table 4-8
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

Shape			2L8×8×												No. of connectors[a]
			$1^1/_8$		1		$^7/_8$		$^3/_4$		$^5/_8$		$^9/_{16}$[c]		
Wt/ft			114		103		90.6		78.4		66.0		59.7		
Design			P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	720	1080	647	972	569	855	491	739	413	621	359	539	b
		2	716	1080	643	967	566	851	489	735	411	618	357	537	
		4	705	1060	634	952	558	838	482	724	405	609	352	529	
		6	687	1030	618	928	544	817	470	706	395	594	344	517	
		8	663	996	596	895	525	789	454	682	382	574	333	500	
		10	632	950	569	855	501	754	434	652	365	549	319	480	
		12	597	897	538	808	474	713	411	617	346	520	303	455	
		14	558	839	503	756	444	667	385	578	325	488	285	428	
		16	516	776	466	700	411	618	357	536	302	453	266	399	
		18	472	710	427	641	377	567	328	493	277	417	245	369	
		20	428	643	387	582	343	515	298	448	253	380	224	337	
		22	384	577	347	522	308	463	268	403	228	342	203	305	
		24	340	512	309	464	274	412	239	359	203	306	182	274	
		26	299	449	272	408	242	363	211	317	180	270	162	244	
		28	259	390	236	355	210	316	184	277	157	236	143	215	
		30	226	339	205	309	183	275	160	241	137	206	124	187	
		32	199	298	181	271	161	242	141	212	120	181	109	164	
		34	176	264	160	240	143	214	125	188	107	160	96.9	146	
		36	157	236	143	214	127	191	111	167	95.1	143	86.5	130	
		38	141	212	128	192	114	172	99.9	150	85.3	128	77.6	117	
		40	127	191	116	174	103	155	90.2	136	77.0	116	70.0	105	
	Y-Y Axis	0	720	1080	647	972	569	855	491	739	413	621	359	539	2
		6	684	1030	609	915	527	793	444	668	332	499	278	417	
		9	665	1000	592	890	513	770	432	650	330	495	276	415	
		12	640	961	569	855	492	740	416	625	325	489	273	410	
		15	608	914	541	813	467	702	395	594	317	477	267	401	
		18	572	859	509	765	438	659	371	558	304	456	258	387	
		21	531	799	473	711	406	611	344	518	285	428	244	367	
		24	488	734	434	653	372	559	316	475	262	394	226	340	
		27	444	667	395	593	337	506	286	430	237	357	206	310	
		30	398	599	354	532	301	452	256	384	212	318	185	279	
		33	354	532	314	472	266	399	226	339	186	280	164	247	
		36	310	466	275	414	231	348	197	296	162	243	143	216	
		39	269	404	238	358	199	299	169	254	139	209	124	186	
		42	232	349	206	309	172	258	146	220	121	181	108	162	
		45	202	304	180	270	150	226	128	192	106	159	94.3	142	
		48	178	268	158	237	132	199	113	169	93.2	140	83.3	125	
		51	158	237	140	211	117	176	100	150	82.8	124	74.1	111	
		54	141	212	125	188	105	157	89.3	134	74.1	111	66.3	99.6	
		57	126	190	112	169	94.0	141	80.3	121	66.6	100	59.6	89.7	
															3

Properties of 2 angles—$^3/_8$ in. back to back

	$1^1/_8$	1	$^7/_8$	$^3/_4$	$^5/_8$	$^9/_{16}$	
A_g (in.^{2})	33.4	30.0	26.4	22.8	19.2	17.4	
r_x (in.)	2.41	2.43	2.45	2.46	2.48	2.49	
r_y (in.)	3.54	3.52	3.50	3.47	3.45	3.44	

Properties of single angle

r_z (in.)	1.56	1.56	1.57	1.57	1.58	1.58	

ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

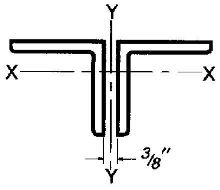
[c] Shape is slender for compression with F_y = 36 ksi.

F_y = 36 ksi

Table 4-8 (continued)

Available Strength in Axial Compression, kips

Double Angles—Equal Legs



2L8–2L6

Shape			2L8×8×		2L6×6×								No. of connectors[a]
			1/2[c]		1		7/8		3/4		5/8		
Wt/ft			53.3		75.0		66.4		57.6		48.5		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	305	458	475	714	421	632	365	548	307	462	b
		2	303	456	471	707	417	626	361	543	305	458	
		4	299	450	458	688	405	609	352	528	297	446	
		6	293	440	436	656	387	581	336	505	284	426	
		8	284	426	409	614	363	545	315	474	266	400	
		10	273	410	375	564	333	501	290	436	246	369	
		12	260	390	338	508	301	453	263	395	223	335	
		14	245	368	299	450	267	401	233	351	198	298	
		16	229	345	260	390	232	349	203	306	173	260	
		18	213	319	221	333	198	298	174	262	149	224	
		20	195	294	185	278	166	250	146	220	125	189	
		22	178	267	153	230	137	207	121	182	104	156	
		24	161	241	128	193	116	174	102	153	87.4	131	
		26	144	216	109	164	98.4	148	86.8	130	74.5	112	
		28	128	192	94.3	142	84.9	128	74.9	113	64.2	96.5	
		30	112	168			73.9	111	65.2	98.0	55.9	84.1	
		32	98.3	148									
		34	87.1	131									
		36	77.7	117									
		38	69.7	105									
		40	62.9	94.6									
	Y-Y Axis	0	305	458	475	714	421	632	365	548	307	462	3
		6	224	337	448	674	394	592	338	507	279	419	
		9	223	335	427	642	375	564	322	483	266	399	
		12	221	332	399	600	351	527	300	452	248	373	
		15	217	326	366	550	321	483	275	414	228	342	
		18	211	317	329	494	288	434	247	371	204	307	
		21	202	304	290	436	254	382	217	327	180	270	
		24	190	285	251	377	219	329	187	282	155	233	
		27	175	263	212	319	185	279	158	238	131	197	
		30	158	238	176	265	153	230	131	196	108	162	
		33	141	212	146	219	127	191	108	163	89.6	135	
		36	125	187	123	184	107	161	91.1	137	75.5	113	
		39	108	163	104	157	91.1	137	77.8	117	64.4	96.9	
		42	94.3	142	90.1	135	78.6	118	67.1	101	55.7	83.6	
		45	82.9	125	78.5	118	68.5	103					
		48	73.3	110									2
		51	65.3	98.1									
		54	58.5	87.9									
		57	52.7	79.2									

Properties of 2 angles—3/8 in. back to back

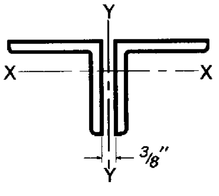
	2L8×8×1/2	2L6×6×1	2L6×6×7/8	2L6×6×3/4	2L6×6×5/8	
A_g (in.^{2})	15.5	22.0	19.5	16.9	14.3	
r_x (in.)	2.49	1.79	1.81	1.82	1.84	
r_y (in.)	3.43	2.72	2.70	2.67	2.65	

Properties of single angle

r_z (in.)	1.59	1.17	1.17	1.17	1.17	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L6

Table 4–8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

Shape		2L6×6×										No. of connectors[a]
		9/16		1/2		7/16[c]		3/8[c]		5/16[c]		
Wt/ft		43.9		39.3		34.6		29.8		25.0		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis — X-X Axis	0	278	418	249	374	213	320	172	259	131	197	b
	2	276	414	247	371	211	317	171	257	130	195	
	4	269	404	240	361	206	309	167	251	127	191	
	6	257	386	230	345	197	297	160	241	123	184	
	8	241	363	216	325	186	279	152	228	117	175	
	10	223	335	200	300	172	259	141	212	110	165	
	12	202	304	181	272	157	236	130	195	101	152	
	14	180	271	162	243	141	211	117	176	92.4	139	
	16	158	237	142	213	124	186	104	156	83.1	125	
	18	135	204	122	183	107	161	90.9	137	73.7	111	
	20	114	172	103	155	91.1	137	78.2	118	64.4	96.8	
	22	95.0	143	85.7	129	76.1	114	66.2	99.5	55.5	83.4	
	24	79.8	120	72.0	108	64.0	96.1	55.6	83.6	47.1	70.7	
	26	68.0	102	61.4	92.2	54.5	81.9	47.4	71.3	40.1	60.3	
	28	58.7	88.2	52.9	79.5	47.0	70.6	40.9	61.4	34.6	52.0	
	30	51.1	76.8	46.1	69.3	40.9	61.5	35.6	53.5	30.1	45.3	
Y-Y Axis	0	278	418	249	374	213	320	172	259	131	197	2
	6	247	372	215	323	166	249	126	189	87.8	132	
	9	236	354	205	309	164	246	124	187	87.0	131	
	12	220	331	192	289	159	238	122	183	85.5	128	
	15	201	302	176	265	149	224	116	175	82.8	125	
	18	180	271	158	238	136	204	108	162	78.6	118	
	21	158	237	139	209	119	180	96.5	145	72.4	109	
	24	135	203	119	179	103	154	84.1	126	64.7	97.3	
	27	113	171	100	150	86.1	129	71.6	108	56.3	84.6	
	30	93.1	140	82.2	124	70.8	106	59.5	89.4	47.8	71.8	
	33	77.3	116	68.3	103	59.0	88.7	49.8	74.9	40.4	60.7	
	36	65.1	97.9	57.7	86.7	49.9	75.1	42.3	63.6	34.5	51.8	
	39	55.6	83.6	49.3	74.1	42.8	64.3	36.3	54.5	29.7	44.6	
	42	48.1	72.2	42.6	64.0	37.0	55.6	31.5	47.3	25.8	38.8	

Properties of 2 angles—3/8 in. back to back

A_g (in.^{2})	12.9	11.5	10.2	8.76	7.34	
r_x (in.)	1.85	1.86	1.86	1.87	1.88	
r_y (in.)	2.64	2.63	2.62	2.60	2.59	

Properties of single angle

r_z (in.)	1.18	1.18	1.18	1.19	1.19	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

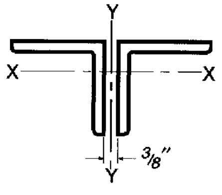
[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

F_y = 36 ksi

Table 4–8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs



2L5

Shape		2L5×5×														No. of connectors[a]
		7/8		3/4		5/8		1/2		7/16		3/8[c]		5/16[c]		
Wt/ft		54.6		47.5		40.1		32.6		28.7		24.8		20.9		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis X-X Axis	0	344	517	299	450	253	380	205	308	180	271	153	230	119	179	b
	2	339	510	295	444	249	375	202	304	178	267	151	227	118	177	
	4	326	490	284	426	240	360	194	292	171	257	146	219	114	171	
	6	304	457	265	398	224	337	182	274	161	241	137	206	107	162	
	8	276	415	241	363	205	308	167	250	147	221	125	189	99.2	149	
	10	244	367	214	321	182	273	148	223	131	197	112	169	89.6	135	
	12	210	316	184	277	157	236	129	193	114	171	97.9	147	79.0	119	
	14	176	264	155	233	133	199	109	164	96.4	145	83.3	125	68.1	102	
	16	143	215	127	190	109	164	89.7	135	79.6	120	69.1	104	57.4	86.3	
	18	114	171	101	152	87.0	131	72.0	108	64.0	96.2	55.8	83.9	47.3	71.1	
	20	92.2	139	81.8	123	70.4	106	58.3	87.6	51.8	77.9	45.2	68.0	38.4	57.6	
	22	76.2	115	67.6	102	58.2	87.5	48.2	72.4	42.8	64.4	37.4	56.2	31.7	47.6	
	24	64.0	96.2	56.8	85.3	48.9	73.5	40.5	60.8	36.0	54.1	31.4	47.2	26.6	40.0	
	26													22.7	34.1	
Y-Y Axis	0	344	517	299	450	253	380	205	308	180	271	153	230	119	179	2
	2	336	505	290	436	242	364	191	287	164	246	122	183	88.5	133	
	4	330	496	285	428	237	357	187	282	161	241	121	183	88.2	133	
	6	320	481	276	415	230	346	182	273	156	234	121	181	87.6	132	
	8	307	461	265	398	221	332	174	262	149	224	119	179	86.6	130	
	10	291	437	251	377	209	314	165	248	141	212	116	174	84.9	128	
	12	272	409	234	352	195	293	154	232	132	198	110	165	82.1	123	
	14	251	378	216	325	180	271	143	214	122	183	103	154	77.9	117	
	16	229	345	197	296	164	247	130	195	111	166	93.6	141	72.3	109	
	18	207	311	178	267	148	222	117	176	99.3	149	84.0	126	65.8	98.9	
	20	184	277	158	237	131	197	104	156	87.8	132	74.2	112	58.9	88.5	
	22	162	243	139	208	115	173	91.1	137	76.6	115	64.6	97.1	51.9	78.0	
	24	141	212	120	181	99.6	150	78.8	118	65.7	98.7	55.3	83.2	45.0	67.6	
	26	121	181	103	155	85.1	128	67.4	101	56.2	84.5	47.5	71.4	38.8	58.3	
	28	104	156	88.8	133	73.5	110	58.2	87.5	48.7	73.1	41.2	61.9	33.8	50.8	
	30	90.7	136	77.4	116	64.1	96.4	50.8	76.4	42.5	63.9	36.1	54.2	29.6	44.5	
	32	79.8	120	68.1	102	56.4	84.8	44.8	67.3	37.4	56.3	31.8	47.8	26.2	39.4	
	34	70.7	106	60.4	90.7	50.0	75.2	39.7	59.7	33.2	49.9	28.3	42.5	23.3	35.0	
	36	63.1	94.8	53.9	81.0	44.6	67.1	35.5	53.3	29.7	44.6	25.3	38.0	20.9	31.4	
	38	56.6	85.1													3

Properties of 2 angles—3/8 in. back to back

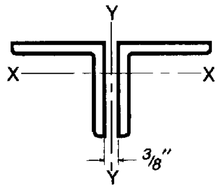
	7/8	3/4	5/8	1/2	7/16	3/8	5/16	
A_g (in.^{2})	16.0	13.9	11.7	9.50	8.36	7.22	6.06	
r_x (in.)	1.49	1.50	1.52	1.53	1.54	1.55	1.56	
r_y (in.)	2.30	2.27	2.25	2.22	2.21	2.20	2.19	

Properties of single angle

r_z (in.)	0.971	0.972	0.975	0.980	0.983	0.986	0.990	

ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L4

Table 4–8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

Shape		2L4×4×														No. of connectors[a]
		3/4		5/8		1/2		7/16		3/8		5/16		1/4[c]		
Wt/ft		37.0		31.3		25.5		22.5		19.4		16.3		13.2		
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis X-X Axis	0	235	353	199	298	162	243	143	214	123	185	103	155	76.2	115	b
	2	230	345	194	292	158	238	140	210	121	182	101	152	74.9	113	
	4	215	323	182	274	149	224	132	198	114	171	95.4	143	71.0	107	
	6	193	290	164	247	134	202	119	179	103	155	86.4	130	64.9	97.6	
	8	166	249	142	213	116	175	103	155	89.5	134	75.2	113	57.3	86.2	
	10	137	205	117	176	96.6	145	85.9	129	74.7	112	63.0	94.7	48.8	73.4	
	12	108	162	92.9	140	77.0	116	68.7	103	59.9	90.0	50.7	76.2	40.1	60.3	
	14	81.3	122	70.5	106	58.8	88.4	52.6	79.1	46.1	69.2	39.1	58.8	31.8	47.8	
	16	62.2	93.5	54.0	81.1	45.0	67.7	40.3	60.6	35.3	53.0	30.0	45.1	24.5	36.9	
	18	49.2	73.9	42.6	64.1	35.6	53.5	31.8	47.8	27.9	41.9	23.7	35.6	19.4	29.1	
	20			34.5	51.9	28.8	43.3	25.8	38.8	22.6	33.9	19.2	28.8	15.7	23.6	
Y-Y Axis	0	235	353	199	298	162	243	143	214	123	185	103	155	76.2	115	3
	2	229	344	192	289	154	232	134	202	113	170	82.8	125	56.1	84.3	
	4	223	335	187	281	150	225	130	196	110	165	82.4	124	55.8	83.9	
	6	213	320	179	268	143	215	125	187	105	158	81.3	122	55.2	83.0	
	8	200	300	167	252	134	201	117	176	98.8	148	78.9	119	54.1	81.4	
	10	184	276	154	231	123	185	107	161	91.0	137	74.3	112	52.1	78.3	
	12	166	250	139	209	111	167	96.9	146	82.2	123	67.8	102	48.7	73.3	
	14	147	222	123	185	98.3	148	85.7	129	72.8	109	60.1	90.3	44.2	66.4	
	16	128	193	107	161	85.3	128	74.4	112	63.2	94.9	52.0	78.1	39.0	58.6	
	18	110	165	91.2	137	72.6	109	63.3	95.1	53.7	80.8	44.0	66.2	33.6	50.5	
	20	92.2	139	76.3	115	60.5	90.9	52.6	79.1	44.7	67.2	36.4	54.8	28.3	42.5	
	22	76.3	115	63.1	94.9	50.1	75.3	43.6	65.6	37.1	55.8	30.4	45.6	23.7	35.7	
	24	64.2	96.5	53.1	79.8	42.2	63.4	36.8	55.2	31.3	47.0	25.7	38.6	20.2	30.3	
	26	54.7	82.2	45.3	68.1	36.0	54.1	31.4	47.2	26.7	40.2	22.0	33.0	17.3	26.0	
	28	47.2	70.9	39.1	58.7	31.0	46.7	27.1	40.7	23.1	34.7	19.0	28.6	15.0	22.6	
	30	41.1	61.8	34.1	51.2	27.1	40.7	23.6	35.5	20.1	30.3					

Properties of 2 angles—3/8 in. back to back

	3/4	5/8	1/2	7/16	3/8	5/16	1/4
A_g (in.^{2})	10.9	9.21	7.49	6.62	5.72	4.80	3.88
r_x (in.)	1.18	1.20	1.21	1.22	1.23	1.24	1.25
r_y (in.)	1.88	1.85	1.83	1.81	1.80	1.79	1.78

Properties of single angle

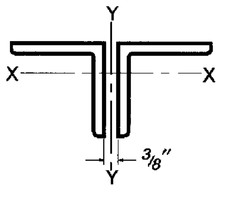
	3/4	5/8	1/2	7/16	3/8	5/16	1/4
r_z (in.)	0.774	0.774	0.776	0.777	0.779	0.781	0.783

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 36 ksi

Table 4-8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs



2L3½

Shape		2L3½×3½×										No. of connectors[a]
		½		7/16		3/8		5/16		¼[c]		
Wt/ft		22.2		19.6		17.0		14.3		11.6		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis												
X-X Axis	0	140	211	124	186	107	161	90.1	135	70.3	106	b
	1	139	209	123	185	106	160	89.5	135	69.9	105	
	2	136	205	120	181	104	156	87.8	132	68.6	103	
	3	132	198	116	175	101	151	85.0	128	66.5	99.9	
	4	126	189	111	167	96.2	145	81.2	122	63.7	95.7	
	5	118	178	105	157	90.6	136	76.5	115	60.2	90.5	
	6	110	165	97.1	146	84.2	127	71.2	107	56.2	84.5	
	7	100	151	89.0	134	77.3	116	65.4	98.3	51.9	77.9	
	8	90.5	136	80.4	121	70.0	105	59.3	89.2	47.2	71.0	
	9	80.6	121	71.8	108	62.5	93.9	53.1	79.8	42.5	63.9	
	10	70.8	106	63.1	94.9	55.1	82.8	46.9	70.5	37.8	56.8	
	11	61.4	92.2	54.8	82.4	47.9	72.1	40.9	61.4	33.2	49.8	
	12	52.3	78.7	46.9	70.5	41.1	61.8	35.2	52.9	28.7	43.2	
	13	44.6	67.0	40.0	60.0	35.0	52.7	30.0	45.0	24.6	36.9	
	14	38.5	57.8	34.4	51.8	30.2	45.4	25.8	38.8	21.2	31.9	
	15	33.5	50.3	30.0	45.1	26.3	39.5	22.5	33.8	18.5	27.7	
	16	29.4	44.2	26.4	39.6	23.1	34.8	19.8	29.7	16.2	24.4	
	17	26.1	39.2	23.4	35.1	20.5	30.8	17.5	26.3	14.4	21.6	
	18							15.6	23.5	12.8	19.3	
Y-Y Axis	0	140	211	124	186	107	161	90.1	135	70.3	106	3
	2	135	203	118	177	100	150	81.7	123	54.8	82.4	
	4	130	195	114	171	96.6	145	79.0	119	54.4	81.8	
	6	122	184	107	161	91.1	137	74.6	112	53.5	80.3	
	8	113	169	98.5	148	83.8	126	68.8	103	51.3	77.2	
	10	101	152	88.3	133	75.2	113	61.9	93.0	47.4	71.2	
	12	88.6	133	77.3	116	65.9	99.0	54.3	81.6	42.0	63.2	
	14	75.7	114	66.0	99.3	56.2	84.5	46.4	69.7	36.1	54.2	
	16	63.1	94.9	55.0	82.7	46.8	70.3	38.6	58.0	30.1	45.2	
	18	51.2	77.0	44.6	67.0	37.9	56.9	31.3	47.0	24.4	36.6	
	20	41.6	62.5	36.2	54.4	30.8	46.3	25.4	38.2	20.0	30.0	
	22	34.4	51.7	30.0	45.0	25.5	38.3	21.1	31.7	16.6	25.0	
	24	28.9	43.5	25.2	37.9	21.5	32.3	17.8	26.7	14.0	21.1	
	26	24.7	37.1	21.5	32.3	18.3	27.5	15.2	22.8	12.0	18.0	

Properties of 2 angles—3/8 in. back to back

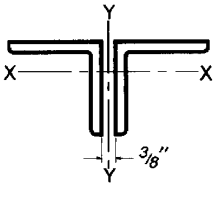
	½	7/16	3/8	5/16	¼	
A_g (in.^{2})	6.50	5.74	4.96	4.18	3.38	
r_x (in.)	1.05	1.06	1.07	1.08	1.09	
r_y (in.)	1.63	1.61	1.60	1.59	1.57	

Properties of single angle

	½	7/16	3/8	5/16	¼	
r_z (in.)	0.679	0.681	0.683	0.685	0.688	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L3

Table 4–8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

Shape			2L3×3×												No. of connectors[a]
			1/2		7/16		3/8		5/16		1/4		3/16[c]		
Wt/ft			18.7		16.6		14.3		12.1		9.77		7.41		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	118	178	105	158	90.9	137	76.6	115	61.9	93.0	42.8	64.3	b
		1	117	176	104	156	90.0	135	75.9	114	61.4	92.2	42.4	63.8	
		2	114	171	101	152	87.6	132	73.8	111	59.8	89.8	41.4	62.3	
		3	109	164	96.4	145	83.7	126	70.6	106	57.2	85.9	39.8	59.8	
		4	102	153	90.3	136	78.5	118	66.3	99.6	53.7	80.8	37.7	56.6	
		5	93.5	141	83.1	125	72.3	109	61.1	91.9	49.6	74.6	35.1	52.7	
		6	84.3	127	75.0	113	65.4	98.2	55.4	83.2	45.0	67.7	32.1	48.3	
		7	74.5	112	66.5	99.9	58.0	87.2	49.3	74.0	40.1	60.3	29.0	43.6	
		8	64.7	97.2	57.8	86.9	50.6	76.0	43.0	64.7	35.1	52.8	25.7	38.7	
		9	55.1	82.8	49.3	74.2	43.3	65.1	36.9	55.5	30.2	45.5	22.5	33.8	
		10	46.0	69.1	41.3	62.1	36.4	54.7	31.1	46.8	25.6	38.4	19.3	29.1	
		11	38.0	57.1	34.2	51.4	30.1	45.3	25.8	38.8	21.2	31.9	16.4	24.6	
		12	31.9	48.0	28.7	43.2	25.3	38.0	21.7	32.6	17.8	26.8	13.7	20.7	
		13	27.2	40.9	24.5	36.8	21.6	32.4	18.5	27.8	15.2	22.8	11.7	17.6	
		14	23.5	35.3	21.1	31.7	18.6	28.0	15.9	24.0	13.1	19.7	10.1	15.2	
		15			18.4	27.6	16.2	24.3	13.9	20.9	11.4	17.2	8.80	13.2	
	Y-Y Axis	0	118	178	105	158	90.9	137	76.6	115	61.9	93.0	42.8	64.3	3
		2	114	172	100	151	86.0	129	70.9	107	54.6	82.1	31.0	46.6	
		4	109	164	95.8	144	82.1	123	67.7	102	52.3	78.6	30.7	46.2	
		6	101	152	88.6	133	75.9	114	62.6	94.1	48.6	73.0	30.0	45.2	
		8	90.4	136	79.3	119	67.9	102	56.1	84.3	43.7	65.7	28.6	43.0	
		10	78.4	118	68.7	103	58.8	88.4	48.6	73.1	38.1	57.2	25.9	39.0	
		12	65.9	99.1	57.7	86.7	49.3	74.1	40.8	61.3	32.0	48.1	22.3	33.6	
		14	53.7	80.7	46.9	70.4	40.0	60.1	33.0	49.7	26.0	39.1	18.4	27.7	
		16	42.2	63.5	36.8	55.3	31.3	47.1	25.9	38.9	20.4	30.7	14.7	22.1	
		18	33.4	50.2	29.1	43.8	24.8	37.3	20.5	30.9	16.2	24.4	11.8	17.7	
		20	27.1	40.7	23.6	35.5	20.1	30.3	16.7	25.1	13.2	19.8	9.65	14.5	
		22	22.4	33.7	19.5	29.4	16.7	25.1	13.8	20.8	10.9	16.5	8.04	12.1	

Properties of 2 angles—3/8 in. back to back

A_g (in.^{2})	5.50	4.86	4.22	3.55	2.87	2.18	
r_x (in.)	0.90	0.90	0.91	0.92	0.93	0.93	
r_y (in.)	1.43	1.42	1.41	1.39	1.38	1.37	

Properties of single angle

r_z (in.)	0.580	0.580	0.581	0.583	0.585	0.586	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

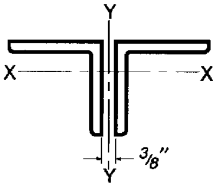
[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 36 ksi

Table 4–8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs



2L2½

Shape		2L2½×2½×										No. of connectors[a]
		½		⅜		$^5/_{16}$		¼		$^3/_{16}$[c]		
Wt/ft		15.3		11.8		10.0		8.07		6.13		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis: X-X Axis	0	96.9	146	74.7	112	63.1	94.8	51.1	76.8	38.1	57.3	b
	1	95.6	144	73.7	111	62.3	93.6	50.5	75.8	37.7	56.6	
	2	91.6	138	70.8	106	59.8	89.9	48.5	72.9	36.3	54.5	
	3	85.4	128	66.2	99.4	56.0	84.2	45.5	68.4	34.1	51.2	
	4	77.5	116	60.2	90.5	51.0	76.7	41.5	62.4	31.2	46.9	
	5	68.3	103	53.3	80.1	45.3	68.1	36.9	55.5	27.9	41.9	
	6	58.5	87.9	45.9	69.0	39.2	58.8	32.0	48.1	24.3	36.5	
	7	48.8	73.3	38.5	57.9	33.0	49.5	27.0	40.6	20.6	31.0	
	8	39.5	59.4	31.5	47.3	27.0	40.6	22.3	33.4	17.1	25.7	
	9	31.3	47.1	25.1	37.7	21.6	32.4	17.8	26.8	13.8	20.7	
	10	25.4	38.1	20.3	30.5	17.5	26.3	14.4	21.7	11.2	16.8	
	11	21.0	31.5	16.8	25.2	14.4	21.7	11.9	17.9	9.23	13.9	
	12	17.6	26.5	14.1	21.2	12.1	18.2	10.0	15.1	7.75	11.7	
Y-Y Axis	0	96.9	146	74.7	112	63.1	94.8	51.1	76.8	38.1	57.3	3
	1	95.2	143	72.5	109	60.4	90.8	47.6	71.5	30.0	45.0	
	2	93.7	141	71.3	107	59.4	89.3	46.8	70.4	29.9	44.9	
	3	91.2	137	69.4	104	57.8	86.9	45.6	68.5	29.7	44.6	
	4	87.8	132	66.8	100	55.6	83.6	43.9	66.0	29.4	44.2	
	5	83.7	126	63.5	95.5	52.9	79.5	41.8	62.8	28.8	43.3	
	6	78.9	119	59.8	89.9	49.8	74.9	39.4	59.2	27.8	41.9	
	7	73.5	111	55.7	83.7	46.3	69.6	36.7	55.1	26.4	39.7	
	8	67.8	102	51.2	77.0	42.6	64.1	33.7	50.7	24.5	36.8	
	9	61.9	93.0	46.6	70.1	38.8	58.3	30.7	46.1	22.4	33.6	
	10	55.8	83.9	42.0	63.1	34.9	52.4	27.6	41.5	20.1	30.3	
	11	49.8	74.9	37.3	56.1	31.0	46.6	24.5	36.9	17.9	26.9	
	12	44.0	66.1	32.9	49.4	27.2	40.9	21.5	32.4	15.7	23.5	
	13	38.4	57.8	28.5	42.9	23.6	35.5	18.6	28.0	13.5	20.4	
	14	33.2	49.9	24.6	37.0	20.4	30.6	16.1	24.2	11.8	17.7	
	15	28.9	43.5	21.5	32.3	17.8	26.7	14.1	21.2	10.3	15.5	
	16	25.4	38.2	18.9	28.4	15.6	23.5	12.4	18.6	9.09	13.7	
	17	22.5	33.9	16.8	25.2	13.9	20.9	11.0	16.5	8.08	12.2	
	18	20.1	30.2	15.0	22.5	12.4	18.6	9.82	14.8	7.23	10.9	
	19	18.1	27.1	13.4	20.2	11.1	16.7	8.83	13.3	6.51	9.78	
	20	16.3	24.5	12.1	18.2							

Properties of 2 angles—⅜ in. back to back

	½	⅜	$^5/_{16}$	¼	$^3/_{16}$	
A_g (in.^{2})	4.50	3.47	2.93	2.37	1.80	
r_x (in.)	0.74	0.75	0.76	0.76	0.77	
r_y (in.)	1.23	1.21	1.19	1.18	1.17	

Properties of single angle

	½	⅜	$^5/_{16}$	¼	$^3/_{16}$	
r_z (in.)	0.481	0.481	0.481	0.482	0.482	

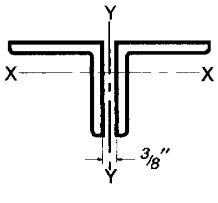
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L2

Table 4–8 (continued)
Available Strength in Axial Compression, kips
Double Angles—Equal Legs

F_y = 36 ksi

Shape		2L2×2×										No. of connectors[a]
		3/8		5/16		1/4		3/16		1/8[c]		
Wt/ft		9.30		7.89		6.43		4.91		3.34		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis X-X Axis	0	58.6	88.1	49.6	74.5	40.4	60.8	30.8	46.3	19.0	28.6	b
	1	57.4	86.2	48.5	73.0	39.6	59.5	30.2	45.4	18.7	28.1	
	2	53.8	80.8	45.5	68.5	37.2	55.9	28.4	42.7	17.7	26.6	
	3	48.2	72.5	41.0	61.6	33.6	50.4	25.7	38.6	16.2	24.3	
	4	41.4	62.2	35.3	53.1	29.0	43.6	22.3	33.5	14.3	21.4	
	5	34.1	51.2	29.2	43.8	24.1	36.2	18.6	28.0	12.1	18.2	
	6	26.8	40.3	23.1	34.7	19.2	28.8	14.9	22.4	10.0	15.0	
	7	20.2	30.4	17.5	26.3	14.6	22.0	11.4	17.2	7.87	11.8	
	8	15.5	23.3	13.4	20.1	11.2	16.8	8.75	13.1	6.06	9.11	
	9	12.2	18.4	10.6	15.9	8.85	13.3	6.91	10.4	4.79	7.20	
	10					7.17	10.8	5.60	8.42	3.88	5.83	
Y-Y Axis	0	58.6	88.1	49.6	74.5	40.4	60.8	30.8	46.3	19.0	28.6	3
	1	57.3	86.2	48.1	72.3	38.7	58.1	28.4	42.6	14.0	21.0	
	2	56.0	84.1	47.0	70.6	37.7	56.7	27.7	41.6	13.9	21.0	
	3	53.8	80.8	45.1	67.8	36.2	54.4	26.6	40.0	13.8	20.8	
	4	50.8	76.4	42.6	64.0	34.2	51.4	25.2	37.8	13.6	20.5	
	5	47.3	71.0	39.6	59.5	31.8	47.7	23.4	35.2	13.2	19.9	
	6	43.2	65.0	36.2	54.3	29.0	43.6	21.4	32.2	12.6	18.9	
	7	38.9	58.5	32.5	48.8	26.0	39.1	19.2	28.9	11.6	17.4	
	8	34.5	51.8	28.7	43.2	23.0	34.5	17.0	25.5	10.4	15.7	
	9	30.1	45.2	25.0	37.5	19.9	30.0	14.7	22.2	9.15	13.8	
	10	25.8	38.7	21.3	32.1	17.0	25.6	12.6	18.9	7.88	11.8	
	11	21.7	32.6	17.9	26.9	14.2	21.4	10.5	15.8	6.66	10.0	
	12	18.2	27.4	15.1	22.6	12.0	18.0	8.85	13.3	5.66	8.51	
	13	15.5	23.4	12.8	19.3	10.2	15.4	7.56	11.4	4.86	7.31	
	14	13.4	20.2	11.1	16.7	8.82	13.3	6.54	9.82	4.22	6.34	
	15	11.7	17.6	9.66	14.5	7.69	11.6	5.70	8.57	3.69	5.55	
	16	10.3	15.4	8.50	12.8	6.77	10.2	5.02	7.54			

Properties of 2 angles—3/8 in. back to back

	3/8	5/16	1/4	3/16	1/8	
A_g (in.^{2})	2.72	2.30	1.88	1.43	0.968	
r_x (in.)	0.59	0.60	0.61	0.61	0.62	
r_y (in.)	1.01	0.996	0.982	0.967	0.951	

Properties of single angle

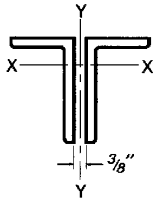
	3/8	5/16	1/4	3/16	1/8	
r_z (in.)	0.386	0.386	0.387	0.389	0.391	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4-9
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L8 LLBB

Shape			2L8×6×													No. of connectors[a]
			1		7/8		3/4		5/8		9/16[c]		1/2[c]		7/16[c]	
Wt/ft			88.8		78.5		68.0		57.3		51.8		46.3		40.7	
Design			P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length KL (ft) with respect to indicated axis	X-X Axis	0	560	842	496	745	429	644	359	540	313	470	265	399	217	327	b
		4	550	826	486	731	420	632	353	530	307	462	261	392	214	322	
		6	536	806	475	713	411	617	345	518	300	451	255	384	210	315	
		8	518	779	459	690	397	597	333	501	291	437	248	373	204	307	
		10	496	745	439	660	380	572	320	480	279	420	239	359	197	296	
		12	470	706	417	626	361	542	304	456	266	400	228	342	189	284	
		14	441	663	391	588	339	510	286	429	251	377	216	324	179	270	
		16	410	616	364	547	316	475	266	400	235	353	202	304	169	254	
		18	377	567	335	504	291	438	246	369	217	327	188	283	158	238	
		20	344	516	306	460	266	400	225	338	200	300	174	261	147	221	
		22	310	466	276	415	241	362	204	306	182	273	159	239	135	203	
		24	277	416	247	372	216	324	183	275	164	246	144	217	124	186	
		26	245	368	219	329	191	288	163	244	146	220	130	195	112	168	
		28	214	322	192	289	168	253	143	215	130	195	116	174	101	151	
		30	187	281	167	252	147	220	125	188	114	171	102	154	89.9	135	
		32	164	247	147	221	129	194	110	165	99.9	150	89.8	135	79.4	119	
		34	145	219	130	196	114	172	97.3	146	88.5	133	79.6	120	70.4	106	
		36	130	195	116	175	102	153	86.7	130	79.0	119	71.0	107	62.8	94.3	
		38	116	175	104	157	91.4	137	77.9	117	70.9	107	63.7	95.7	56.3	84.7	
		40	105	158	94.1	142	82.4	124	70.3	106	64.0	96.1	57.5	86.4	50.8	76.4	
		42					74.8	112	63.7	95.8	58.0	87.2	52.1	78.4	46.1	69.3	
	Y-Y Axis	0	560	842	496	745	429	644	359	540	313	470	265	399	217	327	2
		4	523	786	455	684	383	575	300	451	253	380	206	310	161	241	
		6	510	767	444	667	373	561	297	447	251	377	205	307	159	240	
		8	493	741	429	644	361	543	292	439	247	371	202	303	157	237	
		10	471	708	410	616	345	519	283	426	240	361	197	296	155	232	
		12	446	670	388	583	327	492	271	407	231	347	191	287	150	226	
		14	417	627	363	546	307	461	255	383	219	329	182	274	145	217	
		16	387	581	337	506	285	428	237	356	204	307	171	257	137	207	
		18	355	533	309	464	261	393	217	326	188	283	159	239	129	194	
		20	322	484	280	421	237	357	196	295	171	257	145	219	119	179	
		22	289	434	252	378	213	320	175	264	154	231	132	198	109	163	
		24	257	386	223	336	189	285	155	233	137	205	118	177	98.2	148	
		26	226	339	196	295	166	250	135	203	120	180	104	156	87.7	132	
		28	196	294	170	256	144	217	118	177	104	157	91.1	137	77.4	116	
		30	171	257	149	224	126	190	103	155	91.8	138	80.3	121	68.5	103	
		32	151	226	131	197	111	167	91.2	137	81.3	122	71.2	107	61.0	91.6	
		34	134	201	116	175	98.9	149	81.2	122	72.4	109	63.5	95.5	54.5	82.0	
		36	119	179	104	156	88.4	133	72.7	109	64.9	97.5	57.0	85.7	49.1	73.7	
		38	107	161	93.4	140	79.5	119	65.4	98.3	58.5	87.9	51.4	77.3	44.3	66.6	
		40	96.8	145	84.4	127	71.8	108	59.2	89.0	52.9	79.6	46.6	70.1	40.2	60.5	
		42	87.9	132													

Properties of 2 angles—3/8 in. back to back

	1	7/8	3/4	5/8	9/16	1/2	7/16
A_g (in.^{2})	26.0	23.0	19.9	16.7	15.1	13.5	11.9
r_x (in.)	2.49	2.50	2.52	2.54	2.55	2.55	2.56
r_y (in.)	2.52	2.50	2.47	2.45	2.44	2.43	2.42

Properties of single angle

	1	7/8	3/4	5/8	9/16	1/2	7/16
r_z (in.)	1.28	1.28	1.29	1.29	1.30	1.30	1.31

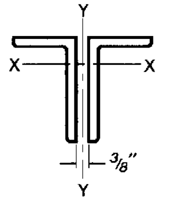
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



2L8 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L8×4×														No. of connectors[a]
		1		7/8		3/4		5/8		9/16[c]		1/2[c]		7/16[c]		
Wt/ft		75.2		66.6		57.8		48.7		44.1		39.5		34.8		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis X-X Axis	0	474	713	419	631	364	547	306	459	266	400	226	340	186	279	b
	4	465	699	412	619	357	537	300	451	261	393	222	334	183	275	
	6	454	683	402	604	349	524	293	441	256	384	218	327	179	269	
	8	439	660	389	584	338	507	284	427	248	372	211	318	174	262	
	10	421	632	373	560	324	487	272	409	238	358	204	306	168	253	
	12	399	600	354	532	307	462	259	389	227	341	195	292	161	243	
	14	375	563	332	500	289	435	244	367	214	322	184	277	154	231	
	16	349	524	310	465	270	405	228	342	201	302	173	260	145	218	
	18	321	483	286	429	249	374	211	316	186	280	161	243	136	204	
	20	293	441	261	392	228	342	193	290	171	257	149	224	126	190	
	22	265	399	236	355	207	310	175	263	156	235	137	205	116	175	
	24	237	357	212	318	185	279	157	237	141	212	124	187	107	160	
	26	211	316	188	283	165	248	140	211	126	190	112	168	96.8	145	
	28	185	278	166	249	145	219	124	186	112	169	100	150	87.2	131	
	30	161	242	144	217	127	191	108	163	98.6	148	88.7	133	78.0	117	
	32	142	213	127	191	111	168	95.2	143	86.6	130	78.0	117	69.1	104	
	34	125	188	112	169	98.7	148	84.3	127	76.7	115	69.1	104	61.2	92.0	
	36	112	168	100	151	88.1	132	75.2	113	68.5	103	61.6	92.6	54.6	82.0	
	38	100	151	89.9	135	79.0	119	67.5	101	61.4	92.3	55.3	83.1	49.0	73.6	
	40	90.6	136	81.2	122	71.3	107	60.9	91.5	55.4	83.3	49.9	75.0	44.2	66.5	
	42			73.6	111	64.7	97.2	55.2	83.0	50.3	75.6	45.3	68.0	40.1	60.3	
Y-Y Axis	0	474	713	419	631	364	547	306	459	266	400	226	340	186	279	2
	4	423	635	366	550	307	462	254	382	215	324	177	266	139	209	
	6	397	597	344	517	289	434	242	363	206	309	170	255	134	201	
	8	364	546	315	473	265	398	222	334	190	286	158	237	126	189	
	10	324	488	281	422	236	355	197	296	170	255	142	214	115	172	
	12	282	424	243	366	205	308	169	254	147	220	124	187	101	152	
	14	238	358	206	309	173	260	141	212	123	185	105	158	86.8	131	
	16	196	295	169	254	142	213	114	171	99.9	150	86.2	130	72.4	109	
	18	166	249	142	213	113	171	91.3	137	80.6	121	69.9	105	59.2	89.0	
	20	135	203	115	173	92.6	139	74.9	113	66.2	99.6	57.7	86.7	49.1	73.8	
	22	112	168	95.7	144	76.9	116	62.4	93.8	55.3	83.2	48.3	72.7	41.3	62.1	
	24	94.1	141	80.6	121	64.9	97.5	52.8	79.3	46.9	70.4	41.0	61.7	35.2	52.9	
	26	80.4	121	68.9	104											3

Properties of 2 angles—3/8 in. back to back

	1	7/8	3/4	5/8	9/16[c]	1/2[c]	7/16[c]	
A_g (in.^{2})	22.0	19.5	16.9	14.2	12.9	11.5	10.1	
r_x (in.)	2.51	2.53	2.55	2.56	2.57	2.58	2.59	
r_y (in.)	1.60	1.57	1.55	1.52	1.51	1.50	1.49	

Properties of single angle

	1	7/8	3/4	5/8	9/16[c]	1/2[c]	7/16[c]	
r_z (in.)	0.844	0.846	0.850	0.856	0.859	0.863	0.867	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

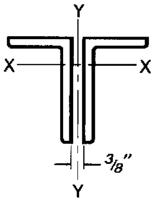
[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L7 LLBB

Shape			2L7×4×										No. of connectors[a]
			3/4		5/8		1/2[c]		7/16[c]		3/8[c]		
Wt/ft			114		103		90.6		78.4		66.0		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	332	498	279	420	218	328	182	273	144	217	b
		4	323	486	273	410	213	321	178	267	141	212	
		6	314	471	264	398	207	312	173	260	138	207	
		8	300	451	253	381	199	299	166	250	133	200	
		10	284	427	240	361	189	284	159	238	127	191	
		12	265	399	224	337	177	267	149	225	121	181	
		14	245	368	207	312	165	247	139	209	113	170	
		16	223	336	189	284	151	227	128	193	105	158	
		18	201	302	171	256	137	206	117	176	96.5	145	
		20	179	269	152	228	122	184	106	159	87.8	132	
		22	157	236	134	201	108	163	94.2	142	79.1	119	
		24	136	205	116	175	95.0	143	83.2	125	70.6	106	
		26	117	175	99.7	150	82.0	123	72.6	109	62.4	93.7	
		28	100	151	86.0	129	70.7	106	62.7	94.3	54.5	81.8	
		30	87.5	132	74.9	113	61.6	92.6	54.6	82.1	47.4	71.3	
		32	76.9	116	65.8	98.9	54.1	81.4	48.0	72.2	41.7	62.7	
		34	68.1	102	58.3	87.6	48.0	72.1	42.5	63.9	36.9	55.5	
		36	60.8	91.3	52.0	78.2	42.8	64.3	37.9	57.0	32.9	49.5	
	Y-Y Axis	0	332	498	279	420	218	328	182	273	144	217	2
		4	291	438	236	355	178	267	142	214	107	161	
		6	274	412	223	335	171	257	137	206	104	156	
		8	252	379	205	308	159	239	129	194	98.7	148	
		10	226	340	184	277	143	215	117	176	91.1	137	
		12	197	297	161	242	125	188	103	156	81.5	123	
		14	168	253	137	206	106	159	88.6	133	70.9	107	
		16	139	210	114	171	87.4	131	73.9	111	60.0	90.3	
		18	112	169	91.7	138	70.5	106	60.1	90.4	49.5	74.5	
		20	91.6	138	74.8	112	57.9	87.1	49.6	74.6	41.2	61.9	
		22	76.0	114	62.2	93.5	48.4	72.7	41.6	62.5	34.7	52.1	
		24	64.0	96.2	52.5	78.9	41.0	61.6	35.3	53.0	29.5	44.4	
		26	54.7	82.2	44.9	67.4	35.1	52.8					

Properties of 2 angles—3/8 in. back to back

	3/4	5/8	1/2	7/16	3/8
A_g (in.^{2})	15.4	13.0	10.5	9.24	7.96
r_x (in.)	2.21	2.23	2.25	2.26	2.27
r_y (in.)	1.61	1.58	1.56	1.55	1.54

Properties of single angle

	3/4	5/8	1/2	7/16	3/8
r_z (in.)	0.855	0.860	0.866	0.869	0.873

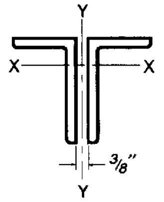
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L6 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape			2L6×4×								No. of connectors[a]
			7/8		3/4		5/8		9/16		
Wt/ft			54.3		47.2		39.9		36.1		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	344	517	299	449	253	380	229	344	b
		4	332	500	289	434	244	367	221	333	
		6	318	478	277	416	234	352	212	319	
		8	299	450	261	392	221	332	200	301	
		10	277	416	241	363	204	307	186	279	
		12	251	378	219	330	186	280	169	254	
		14	224	337	196	295	167	251	152	228	
		16	197	296	172	259	147	221	134	201	
		18	169	255	149	224	127	191	116	175	
		20	144	216	127	190	108	163	99.0	149	
		22	119	179	106	159	90.7	136	82.9	125	
		24	100	151	88.7	133	76.2	115	69.6	105	
		26	85.5	128	75.5	114	64.9	97.6	59.3	89.2	
		28	73.7	111	65.1	97.9	56.0	84.1	51.2	76.9	
		30	64.2	96.5	56.7	85.3	48.8	73.3	44.6	67.0	
	Y-Y Axis	0	344	517	299	449	253	380	229	344	2
		4	317	476	271	407	223	335	198	298	
		6	299	450	256	385	211	317	187	282	
		8	277	416	237	356	195	293	173	260	
		10	250	376	213	321	176	264	157	235	
		12	220	331	188	283	155	233	138	208	
		14	190	286	162	243	133	201	119	179	
		16	160	240	136	204	112	168	99.8	150	
		18	138	207	111	167	91.5	137	81.5	123	
		20	112	169	94.8	143	74.5	112	66.5	99.9	
		22	93.0	140	78.6	118	61.8	92.9	55.2	83.0	
		24	78.3	118	66.2	99.5	52.1	78.3	46.6	70.0	
		26	66.8	100	56.5	84.9	44.5	66.9	39.8	59.8	
		28	57.6	86.6	48.8	73.3					3

Properties of 2 angles—3/8 in. back to back

	7/8	3/4	5/8	9/16
A_g (in.^{2})	16.0	13.9	11.7	10.6
r_x (in.)	1.86	1.88	1.89	1.90
r_y (in.)	1.71	1.68	1.66	1.65

Properties of single angle

	7/8	3/4	5/8	9/16
r_z (in.)	0.854	0.856	0.859	0.861

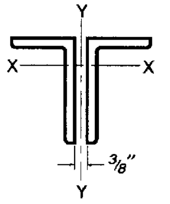
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L6 LLBB

Shape		2L6×4×								No. of connectors[a]
		1/2		7/16[c]		3/8[c]		5/16[c]		
Wt/ft		32.3		28.5		24.6		20.6		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis — X-X Axis	0	205	308	175	264	142	213	108	162	b
	4	198	298	170	255	138	207	105	158	
	6	190	286	163	245	133	199	101	153	
	8	179	269	154	232	126	189	96.8	146	
	10	166	250	144	216	118	177	91.2	137	
	12	152	228	132	198	109	163	84.7	127	
	14	136	205	119	178	98.5	148	77.7	117	
	16	120	181	105	158	88.1	132	70.3	106	
	18	105	157	91.7	138	77.7	117	62.8	94.3	
	20	89.3	134	78.8	118	67.4	101	55.3	83.1	
	22	74.8	112	66.5	99.9	57.7	86.7	48.1	72.2	
	24	62.9	94.5	55.9	84.0	48.6	73.1	41.2	61.9	
	26	53.6	80.5	47.6	71.6	41.4	62.3	35.1	52.7	
	28	46.2	69.4	41.1	61.7	35.7	53.7	30.2	45.5	
	30	40.2	60.5	35.8	53.7	31.1	46.8	26.3	39.6	
	32			31.4	47.2	27.4	41.1	23.2	34.8	
Y-Y Axis	0	205	308	175	264	142	213	108	162	2
	4	172	258	143	215	110	166	78.4	118	
	6	163	245	138	208	107	161	76.6	115	
	8	151	227	130	196	102	153	73.6	111	
	10	137	206	119	178	94.0	141	69.0	104	
	12	121	182	105	157	84.0	126	62.9	94.5	
	14	104	157	89.7	135	73.0	110	55.7	83.8	
	16	87.6	132	75.0	113	61.8	92.8	48.1	72.3	
	18	71.6	108	61.1	91.8	50.9	76.5	40.4	60.7	
	20	58.5	87.9	50.1	75.4	42.0	63.1	33.7	50.7	
	22	48.7	73.1	41.8	62.9	35.2	52.9	28.4	42.7	
	24	41.1	61.7	35.4	53.2	29.9	44.9	24.3	36.5	
	26	35.1	52.8	30.3	45.6	25.6	38.5	20.9	31.4	

Properties of 2 angles—3/8 in. back to back

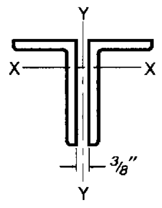
	1/2	7/16	3/8	5/16	
A_g (in.^{2})	9.50	8.36	7.22	6.05	
r_x (in.)	1.91	1.92	1.93	1.94	
r_y (in.)	1.64	1.62	1.61	1.60	

Properties of single angle

	1/2	7/16	3/8	5/16	
r_z (in.)	0.864	0.867	0.870	0.874	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L6 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape			2L6×3½×						No. of connectors[a]
			1/2		3/8[c]		5/16[c]		
Wt/ft			30.7		23.4		19.7		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	194	292	134	202	102	154	b
		2	192	289	133	201	102	153	
		4	188	282	130	196	99.5	150	
		6	180	271	126	189	96.3	145	
		8	170	256	119	179	91.9	138	
		10	158	237	112	168	86.6	130	
		12	144	217	103	155	80.5	121	
		14	129	195	93.6	141	73.8	111	
		16	114	172	83.8	126	66.8	100	
		18	99.4	149	73.9	111	59.7	89.7	
		20	85.0	128	64.2	96.5	52.6	79.1	
		22	71.3	107	55.0	82.6	45.8	68.8	
		24	59.9	90.1	46.4	69.7	39.3	59.0	
		28	44.0	66.2	34.1	51.2	28.8	43.4	
		30	38.4	57.7	29.7	44.6	25.1	37.8	
		32	33.7	50.7	26.1	39.2	22.1	33.2	
	Y-Y Axis	0	194	292	134	202	102	154	2
		2	166	249	107	160	76.0	114	
		4	159	239	104	157	74.6	112	
		6	148	222	99.5	150	71.7	108	
		8	133	200	91.4	137	67.0	101	
		10	116	175	80.6	121	60.3	90.6	
		12	98.0	147	68.5	103	52.3	78.6	
		14	79.9	120	56.1	84.3	43.8	65.8	
		16	62.9	94.5	44.6	67.0	35.5	53.3	
		18	50.2	75.4	35.9	54.0	28.9	43.4	
		20	40.9	61.5	29.5	44.4	23.9	35.9	
		22	34.0	51.1	24.7	37.1	20.0	30.1	

Properties of 2 angles—3/8 in. back to back

	1/2	3/8[c]	5/16[c]
A_g (in.^{2})	9.00	6.84	5.74
r_x (in.)	1.92	1.93	1.94
r_y (in.)	1.40	1.38	1.37

Properties of single angle

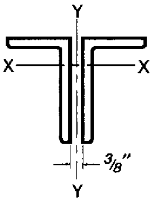
	1/2	3/8[c]	5/16[c]
r_z (in.)	0.756	0.763	0.767

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L5 LLBB

Shape		2L5×3½×										No. of connectors[a]
		3/4		5/8		1/2		3/8[c]		5/16[c]		
Wt/ft		39.6		33.5		27.2		20.8		17.4		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis — X-X Axis	0	250	376	212	319	173	259	129	194	101	151	
	2	247	372	209	315	171	256	128	192	99.6	150	
	4	238	358	202	303	164	247	123	185	96.4	145	
	6	223	336	190	285	155	232	116	175	91.4	137	
	8	204	307	174	261	142	213	107	161	84.8	127	
	10	182	274	155	234	127	191	96.4	145	76.9	116	
	12	159	238	136	204	111	167	84.7	127	68.3	103	b
	14	134	202	115	173	95.0	143	72.7	109	59.4	89.3	
	16	111	167	95.6	144	79.1	119	61.0	91.7	50.6	76.0	
	18	89.4	134	77.2	116	64.2	96.4	49.9	75.0	42.1	63.3	
	20	72.4	109	62.6	94.0	52.0	78.1	40.4	60.8	34.3	51.6	
	22	59.8	89.9	51.7	77.7	43.0	64.6	33.4	50.2	28.4	42.6	
	24	50.3	75.6	43.4	65.3	36.1	54.2	28.1	42.2	23.8	35.8	
Y-Y Axis	0	250	376	212	319	173	259	129	194	101	151	
	2	239	359	199	299	157	235	108	162	79.1	119	
	4	229	344	190	286	150	226	106	159	77.8	117	
	6	213	320	177	267	140	210	101	152	75.1	113	
	8	193	290	160	241	127	190	92.7	139	70.1	105	
	10	170	255	141	212	111	167	81.7	123	62.8	94.4	
	12	145	218	120	180	94.9	143	69.4	104	54.3	81.6	2
	14	120	180	99.3	149	78.4	118	57.0	85.7	45.3	68.1	
	16	101	152	79.6	120	62.7	94.2	45.3	68.1	36.6	55.0	
	18	80.7	121	66.2	99.5	49.8	74.9	36.3	54.5	29.5	44.3	
	20	65.5	98.4	53.8	80.8	40.5	60.9	29.7	44.6	24.2	36.4	
	22	54.2	81.5	44.5	66.9	33.6	50.5	24.7	37.1	20.2	30.4	
	24	45.6	68.5	37.5	56.3	28.3	42.6	20.8	31.3	17.1	25.7	
												3

Properties of 2 angles—3/8 in. back to back

	3/4	5/8	1/2	3/8[c]	5/16[c]
A_g (in.^{2})	11.6	9.84	8.01	6.10	5.12
r_x (in.)	1.55	1.56	1.58	1.59	1.60
r_y (in.)	1.53	1.50	1.48	1.46	1.44

Properties of single angle

	3/4	5/8	1/2	3/8[c]	5/16[c]
r_z (in.)	0.744	0.746	0.750	0.755	0.758

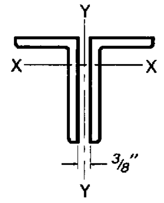
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L5 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L5×3×										No. of connectors[a]
		1/2		7/16		3/8[c]		5/16[c]		1/4[c]		
Wt/ft		25.5		22.5		19.5		16.4		13.2		
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length KL (ft) with respect to indicated axis — X-X Axis	0	162	243	143	215	121	182	94.3	142	67.3	101	
	2	160	240	141	212	120	180	93.3	140	66.7	100	
	4	154	232	136	205	116	174	90.4	136	64.9	97.5	
	6	145	218	128	193	109	164	85.7	129	61.9	93.0	
	8	133	200	118	177	101	151	79.5	120	58.0	87.2	
	10	120	180	106	159	90.7	136	72.3	109	53.3	80.2	
	12	105	157	92.9	140	79.9	120	64.3	96.6	48.1	72.4	
	14	89.6	135	79.5	120	68.7	103	55.9	84.1	42.7	64.1	b
	16	74.7	112	66.5	99.9	57.7	86.7	47.7	71.7	37.1	55.8	
	18	60.7	91.3	54.2	81.4	47.3	71.1	39.8	59.8	31.7	47.6	
	20	49.2	74.0	43.9	66.0	38.3	57.6	32.5	48.8	26.5	39.9	
	22	40.7	61.1	36.3	54.5	31.7	47.6	26.8	40.3	21.9	33.0	
	24	34.2	51.4	30.5	45.8	26.6	40.0	22.6	33.9	18.4	27.7	
Y-Y Axis	0	162	243	143	215	121	182	94.3	142	67.3	101	
	2	145	217	124	186	101	153	74.8	112	49.3	74.1	
	4	136	205	117	176	98.1	147	72.6	109	48.1	72.3	
	6	123	185	106	159	90.3	136	67.8	102	45.7	68.7	
	8	107	161	92.1	138	78.6	118	60.1	90.3	41.6	62.5	
	10	89.0	134	76.7	115	64.9	97.5	50.5	75.9	36.0	54.2	2
	12	70.8	106	61.0	91.8	51.1	76.8	40.5	60.9	29.8	44.8	
	14	56.4	84.7	46.5	69.8	38.7	58.2	31.2	46.8	23.6	35.5	
	16	43.5	65.3	35.9	54.0	30.1	45.2	24.4	36.7	18.7	28.1	
	18	34.5	51.9	28.5	42.9	24.0	36.1	19.6	29.4	15.1	22.7	
	20	28.1	42.2	23.2	34.9	19.6	29.5	16.0	24.0			
												3

Properties of 2 angles—3/8 in. back to back

	1/2	7/16	3/8	5/16	1/4
A_g (in.^{2})	7.51	6.62	5.73	4.80	3.88
r_x (in.)	1.58	1.59	1.60	1.61	1.62
r_y (in.)	1.24	1.23	1.22	1.21	1.19

Properties of single angle

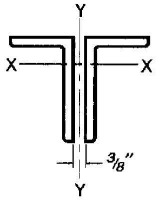
	1/2	7/16	3/8	5/16	1/4
r_z (in.)	0.642	0.644	0.646	0.649	0.652

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L4 LLBB

Shape			2L4×3½×								No. of connectors[a]
			1/2		3/8		5/16		1/4[c]		
Wt/ft			23.8		18.2		15.3		12.4		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	151	227	115	173	96.7	145	71.1	107	b
		2	148	223	113	170	94.9	143	69.9	105	
		4	139	210	106	160	89.6	135	66.4	99.7	
		6	126	190	96.5	145	81.4	122	60.8	91.4	
		8	110	165	84.2	127	71.1	107	53.9	81.0	
		10	91.5	137	70.6	106	59.8	89.9	46.1	69.2	
		12	73.4	110	56.9	85.6	48.4	72.7	38.1	57.2	
		14	56.4	84.8	44.1	66.3	37.6	56.6	30.4	45.6	
		16	43.2	64.9	33.8	50.8	28.8	43.3	23.5	35.3	
		18	34.1	51.3	26.7	40.1	22.8	34.2	18.5	27.9	
		20	27.6	41.5	21.6	32.5	18.4	27.7	15.0	22.6	
	Y-Y Axis	0	151	227	115	173	96.7	145	71.1	107	2
		2	143	215	105	157	79.6	120	54.3	81.7	
		4	137	207	101	151	78.7	118	53.8	80.9	
		6	129	193	94.5	142	76.3	115	52.6	79.0	
		8	117	176	86.4	130	71.3	107	50.1	75.4	
		10	104	157	76.8	115	63.8	95.9	46.0	69.2	
		12	89.9	135	66.4	99.8	55.1	82.8	40.6	61.1	
		14	75.6	114	55.8	83.9	46.1	69.2	34.6	52.1	
		16	61.7	92.8	45.6	68.5	37.3	56.1	28.7	43.1	
		18	51.6	77.5	36.4	54.7	29.8	44.9	23.2	34.8	
		20	41.9	63.0	29.6	44.5	24.4	36.6	19.0	28.6	
		22	34.7	52.1	24.5	36.9	20.3	30.4	15.9	23.9	
		24	29.2	43.9	20.7	31.1	17.1	25.7	13.4	20.2	
		26	24.9	37.4							3

Properties of 2 angles—3/8 in. back to back

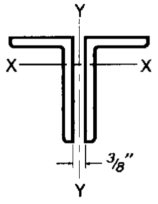
	1/2	3/8	5/16	1/4
A_g (in.^{2})	7.01	5.34	4.50	3.62
r_x (in.)	1.23	1.25	1.25	1.26
r_y (in.)	1.57	1.55	1.53	1.52

Properties of single angle

	1/2	3/8	5/16	1/4
r_z (in.)	0.716	0.719	0.721	0.723

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L4 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape		2L4×3×										No. of connectors[a]
		5/8		1/2		3/8		5/16		1/4[c]		
Wt/ft		27.1		22.1		16.9		14.2		11.5		
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length *KL* (ft) with respect to indicated axis — X-X Axis	0	168	252	140	211	107	161	89.9	135	66.4	99.8	b
	2	164	247	137	207	105	158	88.2	133	65.3	98.1	
	4	155	233	130	195	99.0	149	83.3	125	62.0	93.3	
	6	140	210	117	176	90.0	135	75.8	114	57.0	85.6	
	8	122	183	102	154	78.7	118	66.5	99.9	50.6	76.0	
	10	101	152	85.8	129	66.3	99.6	56.1	84.3	43.4	65.2	
	12	81.3	122	69.1	104	53.7	80.7	45.6	68.5	36.0	54.1	
	14	62.4	93.8	53.5	80.3	41.8	62.9	35.7	53.6	28.8	43.4	
	16	47.8	71.8	40.9	61.5	32.0	48.1	27.3	41.1	22.4	33.6	
	18	37.8	56.8	32.3	48.6	25.3	38.0	21.6	32.5	17.7	26.6	
	20	30.6	46.0	26.2	39.4	20.5	30.8	17.5	26.3	14.3	21.5	
Y-Y Axis	0	168	252	140	211	107	161	89.9	135	66.4	99.8	2
	2	160	241	131	197	96.0	144	75.3	113	52.0	78.2	
	4	152	228	124	187	90.9	137	73.5	110	51.0	76.7	
	6	138	208	113	170	83.0	125	68.8	103	48.5	73.0	
	8	121	182	99.3	149	73.0	110	60.8	91.5	44.0	66.2	
	10	103	154	83.8	126	61.7	92.7	51.2	76.9	37.9	57.0	
	12	83.6	126	68.0	102	50.1	75.2	41.2	61.9	31.1	46.8	
	14	68.9	104	55.6	83.6	38.9	58.5	31.7	47.7	24.5	36.8	
	16	53.2	80.0	42.9	64.4	30.0	45.1	24.6	37.0	19.2	28.8	
	18	42.1	63.3	34.0	51.0	23.8	35.8	19.6	29.4	15.4	23.1	
	20	34.2	51.3	27.6	41.4	19.4	29.1	16.0	24.0	12.6	18.9	
	22	28.3	42.5	22.8	34.3							3

Properties of 2 angles—3/8 in. back to back

	5/8	1/2	3/8	5/16	1/4
A_g (in.^{2})	7.78	6.50	4.96	4.18	3.38
r_x (in.)	1.23	1.24	1.26	1.27	1.27
r_y (in.)	1.35	1.32	1.30	1.29	1.27

Properties of single angle

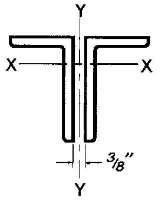
	5/8	1/2	3/8	5/16	1/4
r_z (in.)	0.631	0.633	0.636	0.638	0.639

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L3½ LLBB

Shape		2L3½×3×										No. of connectors[a]
		½		7/16		3/8		5/16		¼[c]		
Wt/ft		20.6		18.2		15.8		13.3		10.8		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis — X-X Axis	0	129	194	114	172	99.2	149	83.2	125	64.9	97.6	b
	2	126	189	111	167	96.6	145	81.1	122	63.4	95.2	
	4	116	175	103	155	89.5	134	75.2	113	58.9	88.6	
	6	102	153	90.3	136	78.6	118	66.2	99.5	52.2	78.5	
	8	84.6	127	75.2	113	65.7	98.7	55.4	83.3	44.1	66.3	
	10	66.7	100	59.4	89.3	52.1	78.3	44.1	66.3	35.5	53.3	
	12	49.7	74.8	44.6	67.0	39.2	59.0	33.4	50.1	27.2	40.9	
	14	36.5	54.9	32.7	49.2	28.8	43.3	24.6	36.9	20.1	30.3	
	16	28.0	42.1	25.1	37.7	22.1	33.2	18.8	28.3	15.4	23.2	
	18			19.8	29.8	17.4	26.2	14.9	22.3	12.2	18.3	
Effective length *KL* (ft) with respect to indicated axis — Y-Y Axis	0	129	194	114	172	99.2	149	83.2	125	64.9	97.6	2
	2	123	185	108	162	91.8	138	74.7	112	52.3	78.6	
	4	117	176	102	154	87.1	131	71.0	107	51.4	77.3	
	6	107	161	93.5	141	79.8	120	65.2	98.0	49.1	73.9	
	8	94.5	142	82.6	124	70.5	106	57.7	86.8	44.6	67.0	
	10	80.5	121	70.3	106	60.1	90.3	49.3	74.0	38.3	57.6	
	12	66.2	99.4	57.7	86.7	49.3	74.1	40.5	60.8	31.5	47.3	
	14	52.4	78.7	45.6	68.5	38.9	58.5	32.0	48.0	24.8	37.3	
	16	42.5	63.9	36.9	55.5	31.4	47.2	24.7	37.1	19.3	29.0	
	18	33.7	50.6	29.2	44.0	24.9	37.5	19.6	29.5	15.4	23.2	
	20	27.3	41.1	23.7	35.7	20.2	30.4	15.9	24.0	12.6	18.9	
	22	22.6	34.0	19.6	29.5	16.8	25.2	13.2	19.9	10.4	15.7	
												3

Properties of 2 angles—3/8 in. back to back

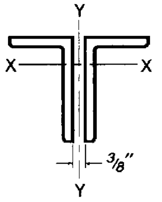
	½	7/16	3/8	5/16	¼
A_g (in.^{2})	6.00	5.30	4.60	3.86	3.12
r_x (in.)	1.07	1.08	1.09	1.09	1.10
r_y (in.)	1.37	1.36	1.35	1.33	1.32

Properties of single angle

	½	7/16	3/8	5/16	¼
r_z (in.)	0.618	0.620	0.622	0.624	0.628

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L3½ LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape			2L3½×2½×								No. of connectors[a]
			½		3/8		5/16		¼[c]		
Wt/ft			18.8		14.5		12.2		9.88		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	119	178	91.0	137	76.7	115	59.9	90.1	b
		1	118	177	90.4	136	76.3	115	59.6	89.5	
		2	116	174	88.7	133	74.9	113	58.5	88.0	
		3	112	168	86.0	129	72.6	109	56.8	85.4	
		4	107	161	82.3	124	69.5	104	54.5	82.0	
		5	101	152	77.8	117	65.7	98.8	51.7	77.7	
		6	94.0	141	72.6	109	61.4	92.3	48.5	72.9	
		7	86.4	130	66.9	101	56.7	85.2	44.9	67.5	
		8	78.4	118	60.9	91.5	51.7	77.6	41.1	61.8	
		9	70.2	106	54.7	82.2	46.5	69.9	37.2	55.9	
		10	62.1	93.4	48.6	73.0	41.3	62.1	33.3	50.0	
		11	54.2	81.5	42.6	64.0	36.3	54.6	29.4	44.2	
		12	46.7	70.3	36.8	55.4	31.5	47.3	25.7	38.6	
		13	39.8	59.9	31.5	47.3	26.9	40.5	22.1	33.3	
		14	34.4	51.6	27.1	40.8	23.2	34.9	19.1	28.7	
		15	29.9	45.0	23.6	35.5	20.2	30.4	16.6	25.0	
		16	26.3	39.5	20.8	31.2	17.8	26.7	14.6	22.0	
		17	23.3	35.0	18.4	27.7	15.8	23.7	12.9	19.4	
		18	20.8	31.2	16.4	24.7	14.1	21.1	11.5	17.3	
	Y-Y Axis	0	119	178	91.0	137	76.7	115	59.9	90.1	2
		1	114	171	84.3	127	68.7	103	49.2	73.9	
		2	111	167	82.7	124	67.4	101	48.9	73.4	
		3	108	162	80.1	120	65.3	98.2	48.2	72.5	
		4	103	155	76.5	115	62.5	93.9	47.0	70.7	
		5	97.1	146	72.2	108	59.0	88.7	45.1	67.9	
		6	90.3	136	67.1	101	55.0	82.6	42.5	63.9	
		7	82.9	125	61.6	92.6	50.5	75.9	39.2	59.0	
		8	75.1	113	55.8	83.8	45.8	68.8	35.6	53.5	
		9	67.1	101	49.8	74.8	40.9	61.5	31.8	47.8	
		10	59.1	88.9	43.8	65.8	36.0	54.1	27.9	42.0	
		11	51.4	77.3	38.0	57.1	31.2	47.0	24.2	36.4	
		12	46.5	69.9	34.1	51.2	26.7	40.1	20.7	31.0	
		13	39.8	59.8	29.1	43.8	23.9	35.9	17.8	26.7	
		14	34.3	51.6	25.2	37.9	20.7	31.1	15.4	23.2	
		15	30.0	45.0	22.0	33.1	18.1	27.2	13.5	20.3	
		16	26.4	39.6	19.4	29.1	15.9	24.0	12.0	18.0	
		17	23.4	35.1	17.2	25.8	14.2	21.3	10.6	16.0	
		18	20.9	31.4	15.4	23.1	12.7	19.0	9.52	14.3	
											3

Properties of 2 angles—3/8 in. back to back					
A_g (in.^{2})	5.50	4.22	3.56	2.88	
r_x (in.)	1.08	1.10	1.11	1.12	
r_y (in.)	1.13	1.11	1.09	1.08	

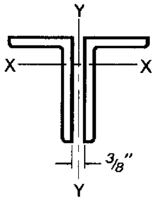
Properties of single angle					
r_z (in.)	0.532	0.535	0.538	0.541	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L3 LLBB

Shape		2L3×2½×												No. of connectors[a]
		½		7/16		3/8		5/16		¼		3/16[c]		
Wt/ft		17.1		15.1		13.1		11.1		8.97		6.82		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	
Effective length *KL* (ft) with respect to indicated axis — **X-X Axis**	0	**108**	162	**95.3**	143	**82.8**	124	**72.0**	108	**56.5**	84.9	**39.1**	58.8	b
	1	**107**	161	**94.4**	142	**82.0**	123	**71.4**	107	**56.0**	84.2	**38.8**	58.4	
	2	**104**	156	**91.9**	138	**79.9**	120	**69.5**	105	**54.6**	82.0	**38.0**	57.0	
	3	**99.3**	149	**87.9**	132	**76.4**	115	**66.6**	100.0	**52.3**	78.6	**36.5**	54.9	
	4	**93.1**	140	**82.5**	124	**71.8**	108	**62.6**	94.1	**49.2**	74.0	**34.6**	52.0	
	5	**85.7**	129	**76.1**	114	**66.3**	99.7	**57.9**	87.0	**45.6**	68.5	**32.3**	48.5	
	6	**77.5**	116	**68.9**	104	**60.2**	90.4	**52.6**	79.0	**41.5**	62.3	**29.7**	44.6	
	7	**68.8**	103	**61.3**	92.1	**53.6**	80.6	**47.0**	70.6	**37.1**	55.8	**26.8**	40.3	
	8	**60.0**	90.1	**53.5**	80.4	**46.9**	70.5	**41.2**	61.9	**32.6**	49.0	**23.9**	35.9	
	9	**51.3**	77.1	**45.9**	69.0	**40.4**	60.7	**35.5**	53.4	**28.2**	42.4	**21.0**	31.5	
	10	**43.1**	64.8	**38.7**	58.1	**34.1**	51.2	**30.1**	45.2	**23.9**	36.0	**18.1**	27.2	
	11	**35.7**	53.6	**32.1**	48.2	**28.3**	42.6	**25.0**	37.6	**20.0**	30.0	**15.4**	23.2	
	12	**30.0**	45.1	**26.9**	40.5	**23.8**	35.8	**21.0**	31.6	**16.8**	25.2	**13.0**	19.5	
	13	**25.6**	38.4	**23.0**	34.5	**20.3**	30.5	**17.9**	26.9	**14.3**	21.5	**11.0**	16.6	
	14	**22.0**	33.1	**19.8**	29.7	**17.5**	26.3	**15.5**	23.2	**12.3**	18.5	**9.52**	14.3	
	15	**19.2**	28.8	**17.2**	25.9	**15.2**	22.9	**13.5**	20.2	**10.7**	16.1	**8.29**	12.5	
Y-Y Axis	0	**108**	162	**95.3**	143	**82.8**	124	**72.0**	108	**56.5**	84.9	**39.1**	58.8	2
	1	**105**	157	**91.9**	138	**78.8**	118	**66.9**	101	**50.2**	75.5	**30.3**	45.5	
	2	**103**	155	**90.3**	136	**77.4**	116	**65.7**	98.7	**49.4**	74.2	**30.1**	45.3	
	3	**100**	150	**87.7**	132	**75.0**	113	**63.7**	95.7	**48.0**	72.1	**29.9**	44.9	
	4	**96.0**	144	**84.2**	127	**71.8**	108	**61.0**	91.7	**46.0**	69.2	**29.4**	44.2	
	5	**91.1**	137	**79.8**	120	**67.9**	102	**57.7**	86.8	**43.6**	65.6	**28.6**	43.0	
	6	**85.5**	128	**74.8**	112	**63.4**	95.3	**53.9**	81.1	**40.9**	61.4	**27.5**	41.3	
	7	**79.2**	119	**69.3**	104	**58.4**	87.8	**49.7**	74.8	**37.8**	56.7	**25.9**	38.9	
	8	**72.6**	109	**63.4**	95.3	**53.2**	79.9	**45.3**	68.1	**34.4**	51.8	**23.9**	35.9	
	9	**65.7**	98.8	**57.3**	86.2	**47.7**	71.8	**40.7**	61.1	**31.0**	46.6	**21.7**	32.6	
	10	**58.8**	88.4	**51.2**	77.0	**42.3**	63.6	**36.1**	54.2	**27.5**	41.3	**19.4**	29.2	
	11	**52.0**	78.1	**45.2**	67.9	**37.0**	55.7	**31.5**	47.4	**24.1**	36.2	**17.1**	25.7	
	12	**45.4**	68.2	**39.4**	59.2	**33.5**	50.4	**27.2**	40.9	**20.7**	31.2	**14.9**	22.3	
	13	**39.1**	58.8	**33.9**	50.9	**28.8**	43.3	**24.4**	36.7	**18.6**	27.9	**12.8**	19.3	
	14	**33.8**	50.7	**29.2**	44.0	**24.9**	37.4	**21.1**	31.7	**16.1**	24.2	**11.2**	16.8	
	15	**29.4**	44.2	**25.5**	38.3	**21.7**	32.6	**18.4**	27.7	**14.1**	21.2	**9.81**	14.7	
	16	**25.9**	38.9	**22.4**	33.7	**19.1**	28.7	**16.2**	24.4	**12.4**	18.6	**8.68**	13.0	
	17	**22.9**	34.5	**19.9**	29.9	**16.9**	25.4	**14.4**	21.6	**11.0**	16.6	**7.72**	11.6	
	18	**20.5**	30.8	**17.8**	26.7	**15.1**	22.7	**12.9**	19.3	**9.85**	14.8	**6.92**	10.4	
	19	**18.4**	27.6	**15.9**	24.0	**13.6**	20.4	**11.6**	17.4					3

Properties of 2 angles—3/8 in. back to back

	½	7/16	3/8	5/16	¼	3/16	
A_g (in.²)	5.00	4.42	3.84	3.34	2.62	1.99	
r_x (in.)	0.910	0.917	0.924	0.932	0.940	0.947	
r_y (in.)	1.18	1.16	1.15	1.14	1.12	1.11	

Properties of single angle

	½	7/16	3/8	5/16	¼	3/16	
r_z (in.)	0.516	0.516	0.517	0.518	0.520	0.521	

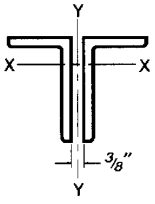
ASD	**LRFD**
Ω_c = 1.67	ϕ_c = 0.90

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



2L3 LLBB

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB

F_y = 36 ksi

Shape			2L3×2×										No. of connectors[a]
			1/2		3/8		5/16		1/4		3/16[c]		
Wt/ft			15.4		11.9		10.1		8.18		6.24		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	97.0	146	74.6	112	62.9	94.6	51.3	77.1	35.4	53.3	b
		1	96.1	145	73.9	111	62.4	93.8	50.9	76.5	35.2	52.9	
		2	93.6	141	72.1	108	60.8	91.5	49.6	74.6	34.4	51.7	
		3	89.5	135	69.0	104	58.3	87.7	47.6	71.5	33.1	49.8	
		4	84.1	126	65.0	97.6	55.0	82.6	44.9	67.5	31.5	47.3	
		5	77.6	117	60.1	90.3	50.9	76.5	41.6	62.6	29.4	44.2	
		6	70.4	106	54.7	82.2	46.4	69.7	38.0	57.1	27.1	40.7	
		7	62.7	94.2	48.9	73.4	41.5	62.4	34.1	51.2	24.6	36.9	
		8	54.8	82.4	42.9	64.5	36.6	55.0	30.1	45.2	22.0	33.0	
		9	47.1	70.8	37.1	55.7	31.7	47.6	26.1	39.2	19.3	29.1	
		10	39.8	59.8	31.5	47.3	26.9	40.5	22.3	33.5	16.8	25.2	
		11	33.0	49.6	26.2	39.4	22.5	33.8	18.7	28.0	14.3	21.6	
		12	27.7	41.7	22.0	33.1	18.9	28.4	15.7	23.6	12.1	18.2	
		13	23.6	35.5	18.8	28.2	16.1	24.2	13.4	20.1	10.3	15.5	
		14	20.4	30.6	16.2	24.3	13.9	20.9	11.5	17.3	8.88	13.3	
		15	17.7	26.7	14.1	21.2	12.1	18.2	10.0	15.1	7.74	11.6	
		16									6.80	10.2	
	Y-Y Axis	0	97.0	146	74.6	112	62.9	94.6	51.3	77.1	35.4	53.3	2
		1	93.5	141	70.2	106	57.8	86.9	44.9	67.5	28.3	42.5	
		2	90.8	136	68.2	102	56.1	84.4	43.7	65.6	27.9	42.0	
		3	86.4	130	64.8	97.4	53.4	80.3	41.6	62.6	27.3	41.0	
		4	80.6	121	60.4	90.8	49.8	74.9	38.9	58.5	26.2	39.3	
		5	73.7	111	55.2	82.9	45.5	68.4	35.6	53.6	24.4	36.7	
		6	66.1	99.4	49.4	74.2	40.7	61.2	31.9	48.0	22.2	33.3	
		7	58.1	87.3	43.2	65.0	35.6	53.6	28.0	42.1	19.6	29.4	
		8	50.0	75.2	37.1	55.7	30.5	45.9	24.0	36.1	16.9	25.4	
		9	44.6	67.0	32.7	49.1	25.6	38.5	20.2	30.3	14.3	21.5	
		10	37.3	56.1	27.1	40.7	22.2	33.3	17.4	26.1	11.8	17.8	
		11	30.9	46.4	22.5	33.8	18.4	27.6	14.5	21.7	9.9	14.9	
		12	26.0	39.1	18.9	28.4	15.5	23.3	12.2	18.4	8.41	12.6	
		13	22.2	33.3	16.2	24.3	13.2	19.9	10.5	15.7	7.22	10.9	
		14	19.1	28.7	14.0	21.0	11.4	17.2	9.04	13.6	6.27	9.42	
		15	16.7	25.1	12.2	18.3							3

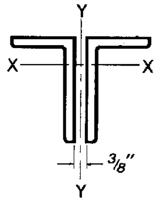
Properties of 2 angles—3/8 in. back to back						
A_g (in.^{2})	4.50	3.46	2.92	2.38	1.80	
r_x (in.)	0.922	0.937	0.945	0.953	0.961	
r_y (in.)	0.940	0.911	0.897	0.883	0.869	
Properties of single angle						
r_z (in.)	0.425	0.426	0.428	0.431	0.435	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4-9 (continued)
Available Strength in Axial Compression, kips
Double Angles—LLBB



2L2½ LLBB

Shape		2L2½×2×								No. of connectors[a]
		3/8		5/16		1/4		3/16[c]		
Wt/ft		10.6		8.97		7.30		5.57		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis: X-X Axis	0	66.8	100	56.5	84.9	45.7	68.7	34.3	51.5	b
	1	66.0	99.2	55.8	83.8	45.1	67.8	33.9	50.9	
	2	63.5	95.4	53.7	80.7	43.5	65.4	32.7	49.1	
	3	59.5	89.4	50.4	75.7	40.9	61.4	30.8	46.3	
	4	54.4	81.7	46.1	69.3	37.5	56.3	28.3	42.6	
	5	48.4	72.7	41.2	61.9	33.5	50.4	25.4	38.2	
	6	42.0	63.1	35.8	53.8	29.3	44.0	22.3	33.5	
	7	35.5	53.3	30.4	45.7	24.9	37.4	19.1	28.7	
	8	29.2	44.0	25.1	37.8	20.7	31.1	16.0	24.0	
	9	23.5	35.2	20.2	30.4	16.7	25.1	13.0	19.6	
	10	19.0	28.5	16.4	24.6	13.5	20.3	10.5	15.9	
	11	15.7	23.6	13.5	20.3	11.2	16.8	8.72	13.1	
	12	13.2	19.8	11.4	17.1	9.40	14.1	7.33	11.0	
	13					8.01	12.0	6.24	9.38	
Effective length *KL* (ft) with respect to indicated axis: Y-Y Axis	0	66.8	100	56.5	84.9	45.7	68.7	34.3	51.5	2
	1	64.3	96.6	53.5	80.4	42.0	63.2	28.1	42.2	
	2	62.6	94.1	52.0	78.2	40.9	61.5	27.8	41.9	
	3	60.0	90.1	49.7	74.6	39.1	58.7	27.3	41.1	
	4	56.4	84.7	46.5	69.9	36.6	55.0	26.3	39.5	
	5	52.1	78.3	42.7	64.2	33.7	50.6	24.6	36.9	
	6	47.3	71.1	38.5	57.9	30.4	45.7	22.3	33.6	
	7	42.1	63.3	34.1	51.2	26.9	40.4	19.8	29.7	
	8	36.9	55.5	29.5	44.4	23.3	35.0	17.1	25.7	
	9	31.7	47.7	25.1	37.7	19.8	29.7	14.5	21.7	
	10	26.8	40.3	22.1	33.2	17.3	26.0	12.6	18.9	3
	11	22.2	33.4	18.3	27.5	14.4	21.6	10.5	15.8	
	12	18.7	28.1	15.4	23.2	12.1	18.2	8.88	13.3	
	13	16.0	24.0	13.2	19.8	10.4	15.6	7.61	11.4	
	14	13.8	20.7	11.4	17.1	8.95	13.5	6.59	9.90	
	15	12.0	18.1	9.91	14.9	7.81	11.7	5.76	8.66	

Properties of 2 angles—3/8 in. back to back

	3/8	5/16	1/4	3/16
A_g (in.^{2})	3.10	2.62	2.12	1.62
r_x (in.)	0.766	0.774	0.782	0.790
r_y (in.)	0.957	0.943	0.930	0.916

Properties of single angle

	3/8	5/16	1/4	3/16
r_z (in.)	0.419	0.420	0.423	0.426

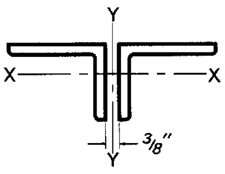
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L8 SLBB

Table 4–10
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape			2L8×6×														No. of connectors[a]
			1		7/8		3/4		5/8		9/16[c]		1/2[c]		7/16[c]		
Wt/ft			88.8		78.5		68.0		57.3		51.8		46.3		40.7		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	560	842	496	745	429	644	359	540	313	470	265	399	217	327	b
		4	538	809	476	716	412	619	346	520	301	453	256	385	211	316	
		6	511	769	453	681	392	589	330	495	288	433	245	369	202	304	
		8	476	716	422	635	366	550	308	463	270	406	231	347	191	288	
		10	434	653	386	580	335	504	282	424	248	373	214	321	178	268	
		12	388	583	346	519	301	452	254	382	225	338	194	292	163	245	
		14	340	511	303	456	264	398	224	337	199	299	174	261	147	221	
		16	292	439	261	392	228	343	194	291	174	261	152	229	130	196	
		18	245	369	220	331	193	290	165	247	148	223	132	198	114	171	
		20	202	303	182	273	160	240	137	206	125	187	112	168	97.7	147	
		22	167	251	150	225	132	198	113	170	103	155	93.0	140	82.4	124	
		24	140	211	126	189	111	167	95.0	143	86.7	130	78.1	117	69.3	104	
		26	119	179	107	161	94.5	142	80.9	122	73.9	111	66.6	100	59.0	88.7	
		28	103	155	92.6	139	81.5	122	69.8	105	63.7	95.7	57.4	86.3	50.9	76.5	
		30													44.3	66.6	
	Y-Y Axis	0	560	842	496	745	429	644	359	540	313	470	265	399	217	327	3
		4	547	822	481	723	411	618	299	449	252	378	205	308	159	239	
		6	541	813	476	715	407	611	299	449	251	378	205	308	159	239	
		8	533	801	468	704	401	602	298	448	251	377	204	307	159	239	
		10	522	785	459	690	393	590	297	447	250	376	204	307	159	238	
		12	509	766	448	673	383	576	296	445	249	375	203	306	158	238	
		16	478	719	421	632	360	541	290	435	246	369	201	302	157	236	
		20	441	663	388	583	332	499	274	412	235	354	195	293	154	231	
		24	400	601	351	528	301	452	250	376	217	327	183	276	148	222	
		28	356	535	312	469	267	401	222	334	195	293	167	250	137	206	
		32	311	467	272	410	233	350	194	291	171	257	147	222	123	185	
		36	266	401	233	351	200	300	166	249	147	221	128	192	108	162	
		40	224	337	196	295	168	252	139	208	124	186	109	163	93.1	140	
		44	186	280	163	245	139	209	115	173	103	155	90.9	137	78.6	118	
		48	156	235	137	206	117	176	96.8	146	86.8	131	76.8	115	66.5	100	
		52	133	201	117	175	99.7	150	82.7	124	74.2	112	65.6	98.6	57.0	85.6	
		56	115	173	101	151	86.1	129	71.4	107	64.1	96.3	56.7	85.3	49.3	74.1	
		60	100	151	87.8	132	75.0	113	62.3	93.6	55.9	84.1	49.5	74.4	43.1	64.8	

Properties of 2 angles—3/8 in. back to back

A_g (in.^{2})	26.0	23.0	19.9	16.7	15.1	13.5	11.9	
r_x (in.)	1.72	1.74	1.75	1.77	1.78	1.79	1.80	
r_y (in.)	3.77	3.75	3.72	3.70	3.69	3.68	3.66	

Properties of single angle

r_z (in.)	1.28	1.28	1.29	1.29	1.30	1.30	1.31	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

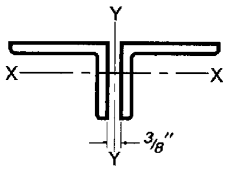
[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L8 SLBB

Shape			2L8×4×														No. of connectors[a]
			1		7/8		3/4		5/8		9/16^c		1/2^c		7/16^c		
Wt/ft			75.2		66.6		57.8		48.7		44.1		39.5		34.8		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	474	713	419	631	364	547	306	459	266	400	226	340	186	279	b
		4	423	635	375	563	326	490	275	413	240	361	205	309	170	255	
		6	366	550	326	489	284	427	240	361	212	318	182	274	152	229	
		8	299	450	267	402	234	352	199	300	177	266	155	232	131	197	
		10	231	347	207	312	183	275	157	236	141	212	125	188	107	161	
		12	168	252	152	228	135	203	117	175	107	160	96.1	144	84.5	127	
		14	123	185	112	168	99.2	149	85.7	129	78.6	118	71.3	107	63.6	95.6	
		16	94.5	142	85.4	128	75.9	114	65.6	98.6	60.2	90.4	54.6	82.0	48.7	73.2	
		18											43.1	64.8	38.5	57.8	
	Y-Y Axis	0	474	713	419	631	364	547	306	459	266	400	226	340	186	279	6
		4	469	705	414	623	358	539	257	386	217	325	177	266	138	208	
		6	465	699	410	617	355	534	257	386	217	325	177	266	138	208	
		8	459	690	405	609	350	526	256	385	216	325	177	266	138	208	
		10	451	678	398	599	344	518	256	385	216	325	177	266	138	208	
		12	442	664	390	586	337	507	256	385	216	325	177	266	138	208	
		16	420	631	370	556	320	481	255	383	215	324	176	265	138	207	
		20	392	590	346	520	299	449	247	372	212	319	175	263	137	206	
		24	362	543	318	479	275	413	229	345	200	301	169	255	135	204	
		28	328	493	289	434	249	374	208	313	183	274	157	236	129	194	
		32	293	441	258	388	222	334	185	279	163	246	141	213	118	178	
		36	259	389	227	341	195	293	163	244	144	217	125	188	106	160	
		40	224	337	197	295	169	254	140	211	125	188	110	165	93.6	141	
		44	192	288	168	252	144	216	119	179	107	161	94.3	142	81.4	122	
		48	162	243	141	212	121	182	100	151	90.1	135	79.9	120	69.6	105	
		52	138	207	120	181	103	155	85.6	129	76.8	115	68.2	102	59.4	89.3	
		56	119	179	104	156	88.9	134	73.8	111	66.3	99.6	58.8	88.4	51.3	77.1	
		60	104	156	90.4	136	77.4	116	64.3	96.7	57.8	86.8	51.3	77.1	44.7	67.2	
		64	91.0	137	79.5	119	68.1	102	56.6	85.0	50.8	76.3	45.1	67.8	39.4	59.1	
		68	80.6	121													

Properties of 2 angles—3/8 in. back to back

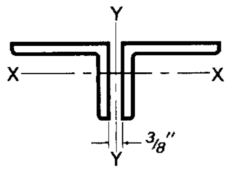
A_g (in.^{2})	22.0	19.5	16.9	14.2	12.9	11.5	10.1	
r_x (in.)	1.03	1.04	1.05	1.06	1.07	1.08	1.09	
r_y (in.)	4.08	4.06	4.03	4.00	3.99	3.97	3.96	

Properties of single angle

r_z (in.)	0.844	0.846	0.850	0.856	0.859	0.863	0.867	

ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



2L7 SLBB

Table 4–10 (continued)
Available Strength in Axial Compression, kips

F_y = 36 ksi

Double Angles—SLBB

Shape			2L7×4×										No. of connectors[a]
			$^3/_4$		$^5/_8$		$^1/_2$[c]		$^7/_{16}$[c]		$^3/_8$[c]		
Wt/ft			52.4		44.2		35.8		31.5		27.2		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	332	498	279	420	218	328	182	273	144	217	b
		4	299	449	253	380	199	299	166	250	133	200	
		6	263	395	223	334	176	265	149	224	120	181	
		8	219	329	186	280	149	224	127	191	104	157	
		10	173	261	148	223	121	181	104	157	87.2	131	
		12	130	196	112	169	92.9	140	81.8	123	69.8	105	
		14	95.8	144	82.8	124	68.9	103	61.4	92.3	53.6	80.6	
		16	73.3	110	63.4	95.2	52.7	79.2	47.0	70.7	41.1	61.7	
		18	57.9	87.1	50.1	75.2	41.7	62.6	37.2	55.9	32.5	48.8	
	Y-Y Axis	0	332	498	279	420	218	328	182	273	144	217	5
		4	325	489	273	410	179	268	142	214	107	160	
		6	321	483	269	404	179	268	142	214	107	160	
		8	316	474	264	397	178	268	142	213	106	160	
		10	308	463	258	388	178	268	142	213	106	160	
		12	300	451	251	377	178	267	142	213	106	160	
		16	279	419	234	351	175	264	140	211	106	159	
		20	254	382	213	320	165	249	136	204	104	157	
		24	227	342	190	285	149	224	125	188	99.3	149	
		28	199	299	166	249	131	197	111	167	90.0	135	
		32	170	256	142	213	112	169	96.0	144	79.2	119	
		36	143	215	119	179	94.4	142	81.5	122	68.1	102	
		40	117	176	97.5	147	77.6	117	67.6	102	57.4	86.3	
		44	97.1	146	80.6	121	64.2	96.5	56.0	84.2	47.7	71.7	
		48	81.6	123	67.8	102	54.0	81.2	47.1	70.8	40.2	60.4	
		52	69.5	105	57.8	86.8	46.1	69.3	40.2	60.4	34.3	51.5	
		56	60.0	90.1	49.8	74.9	39.8	59.8	34.7	52.1	29.6	44.5	

Properties of 2 angles—$^3/_8$ in. back to back

A_g (in.^{2})	15.4	13.0	10.5	9.24	7.96	
r_x (in.)	1.08	1.10	1.11	1.12	1.12	
r_y (in.)	3.48	3.46	3.43	3.42	3.40	

Properties of single angle

r_z (in.)	0.855	0.860	0.866	0.869	0.873	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

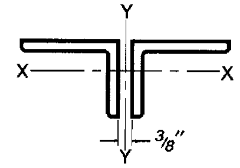
[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L6 SLBB

Shape			2L6×4×								No. of connectors[a]
			7/8		3/4		5/8		9/16		
Wt/ft			54.3		47.2		39.9		36.1		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	344	517	299	449	253	380	229	344	b
		4	311	468	271	408	230	345	208	313	
		6	275	413	240	361	204	307	185	278	
		8	231	347	203	304	173	260	157	236	
		10	184	277	163	244	139	210	127	191	
		12	140	211	124	187	107	161	98.4	148	
		14	103	155	92.0	138	79.6	120	73.1	110	
		16	79.1	119	70.4	106	61.0	91.6	56.0	84.1	
		18	62.5	94.0	55.6	83.6	48.2	72.4	44.2	66.5	
	Y-Y Axis	0	344	517	299	449	253	380	229	344	4
		4	337	507	292	439	245	368	221	332	
		6	331	497	287	431	240	361	217	326	
		8	323	485	279	420	234	352	211	317	
		10	312	470	270	406	227	341	204	307	
		12	300	451	260	390	218	327	196	295	
		16	271	408	235	352	197	295	177	266	
		20	238	358	206	309	172	259	155	233	
		24	203	306	175	263	146	220	132	198	
		28	169	253	145	218	121	181	109	163	
		32	136	204	116	175	96.5	145	86.7	130	
		36	107	161	91.8	138	76.3	115	68.6	103	
		40	87.0	131	74.4	112	61.9	93.0	55.6	83.6	
		44	71.9	108	61.5	92.5	51.2	76.9	46.0	69.1	
		48	60.4	90.8	51.7	77.7	43.0	64.7	38.7	58.1	

Properties of 2 angles—3/8 in. back to back

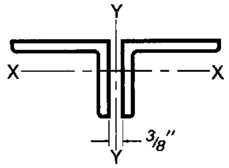
A_g (in.^{2})	16.0	13.9	11.7	10.6	
r_x (in.)	1.10	1.12	1.13	1.14	
r_y (in.)	2.96	2.94	2.91	2.90	

Properties of single angle

r_z (in.)	0.854	0.856	0.859	0.861	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L6 SLBB

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape		2L6×4×								No. of connectors[a]
		1/2		7/16[c]		3/8[c]		5/16[c]		
Wt/ft		32.3		28.5		24.6		20.6		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length KL (ft) with respect to indicated axis X-X Axis	0	205	308	175	264	142	213	108	162	b
	4	187	281	161	241	131	196	100	150	
	6	166	250	144	216	118	177	91.3	137	
	8	141	213	123	185	102	153	80.3	121	
	10	115	173	101	151	84.9	128	68.1	102	
	12	89.0	134	78.9	119	67.7	102	55.7	83.7	
	14	66.3	99.6	59.2	88.9	51.8	77.8	43.8	65.9	
	16	50.8	76.3	45.3	68.1	39.6	59.6	33.7	50.6	
	18	40.1	60.3	35.8	53.8	31.3	47.1	26.6	40.0	
Y-Y Axis	0	205	308	175	264	142	213	108	162	4
	4	196	294	143	215	110	165	77.8	117	
	6	192	289	143	215	110	165	77.7	117	
	8	187	282	143	215	110	165	77.6	117	
	10	181	273	142	214	109	164	77.4	116	
	12	174	262	141	212	109	163	77.1	116	
	16	157	236	133	201	105	158	75.9	114	
	20	138	207	119	178	96.3	145	72.3	109	
	24	117	176	101	152	83.5	125	64.9	97.5	
	28	96.3	145	83.5	126	69.9	105	55.6	83.6	
	32	76.8	115	66.8	100	56.7	85.3	46.2	69.4	
	36	60.8	91.4	53.0	79.6	45.1	67.8	37.2	55.9	
	40	49.3	74.2	43.0	64.6	36.7	55.1	30.3	45.6	
	44	40.8	61.3	35.6	53.5	30.4	45.7	25.2	37.8	
	48	34.3	51.6	29.9	45.0					

Properties of 2 angles—3/8 in. back to back

	1/2	7/16[c]	3/8[c]	5/16[c]
A_g (in.^{2})	9.50	8.36	7.22	6.05
r_x (in.)	1.14	1.15	1.16	1.17
r_y (in.)	2.89	2.88	2.86	2.85

Properties of single angle

	1/2	7/16[c]	3/8[c]	5/16[c]
r_z (in.)	0.864	0.867	0.870	0.874

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

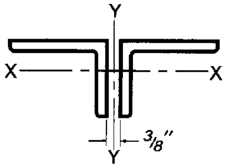
[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L6 SLBB

Shape		2L6×3½×						
		½		$^3/_8$[c]		$^5/_{16}$[c]		No. of connectors[a]
Wt/ft		30.7		23.4		19.7		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis — **X-X Axis**	0	194	292	134	202	102	154	b
	1	192	289	133	201	102	153	
	2	188	282	131	196	99.6	150	
	3	180	271	126	189	96.5	145	
	4	170	256	120	180	92.3	139	
	5	159	238	112	169	87.1	131	
	6	145	218	104	156	81.2	122	
	7	131	196	94.7	142	74.8	112	
	8	116	174	85.1	128	68.0	102	
	9	101	151	75.4	113	61.0	91.7	
	10	86.4	130	65.8	98.9	54.0	81.2	
	11	72.8	109	56.6	85.1	47.3	71.1	
	12	61.2	91.9	48.0	72.1	40.8	61.4	
	13	52.1	78.3	40.9	61.4	34.8	52.4	
	14	44.9	67.5	35.2	53.0	30.0	45.1	
	15	39.1	58.8	30.7	46.1	26.2	39.3	
	16	34.4	51.7	27.0	40.5	23.0	34.6	
Y-Y Axis	0	194	292	134	202	102	154	5
	6	184	277	105	157	74.3	112	
	8	180	271	105	157	74.2	111	
	10	174	262	104	157	74.1	111	
	12	168	252	104	156	73.9	111	
	14	161	241	103	155	73.6	111	
	16	152	229	102	153	73.2	110	
	18	144	216	98.9	149	72.3	109	
	20	134	202	94.1	141	70.6	106	
	22	125	187	88.3	133	67.8	102	
	24	115	173	82.1	123	64.0	96.2	
	26	105	158	75.8	114	59.7	89.8	
	28	95.8	144	69.4	104	55.3	83.1	
	30	86.4	130	63.1	94.9	50.8	76.3	
	32	77.5	116	57.0	85.7	46.3	69.6	
	34	68.8	103	51.1	76.8	42.0	63.1	
	38	55.1	82.9	41.0	61.7	33.9	51.0	
	42	45.2	67.9	33.6	50.6	27.9	41.9	
	46	37.7	56.6	28.1	42.2	23.3	35.0	
	48	34.6	52.0	25.8	38.8	21.4	32.2	

Properties of 2 angles—$^3/_8$ in. back to back

	½	$^3/_8$	$^5/_{16}$	
A_g (in.^{2})	9.00	6.84	5.74	
r_x (in.)	0.968	0.984	0.991	
r_y (in.)	2.96	2.94	2.92	

Properties of single angle

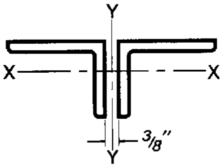
	½	$^3/_8$	$^5/_{16}$	
r_z (in.)	0.756	0.763	0.767	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.



2L5 SLBB

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape		2L5×3½×										No. of connectors[a]
		3/4		5/8		1/2		3/8[c]		5/16[c]		
Wt/ft		39.6		33.5		27.2		20.8		17.4		
Design		P_n/Ω_c ASD	ϕ_cP_n LRFD	P_n/Ω_c ASD	ϕ_cP_n LRFD	P_n/Ω_c ASD	ϕ_cP_n LRFD	P_n/Ω_c ASD	ϕ_cP_n LRFD	P_n/Ω_c ASD	ϕ_cP_n LRFD	
Effective length KL (ft) with respect to indicated axis: X-X Axis	0	250	376	212	319	173	259	129	194	101	151	b
	1	248	373	210	316	171	257	128	193	100	150	
	2	243	365	206	309	167	252	126	189	98.1	147	
	3	233	350	198	297	161	242	121	182	94.9	143	
	4	220	331	187	282	153	230	115	173	90.6	136	
	5	205	308	175	262	143	215	108	162	85.4	128	
	6	188	282	160	241	132	198	99.7	150	79.4	119	
	7	169	255	145	218	119	179	90.8	136	72.9	110	
	8	150	226	129	194	106	160	81.5	123	66.1	99.3	
	9	131	197	113	170	93.6	141	72.1	108	59.1	88.8	
	10	113	169	97.5	147	81.1	122	62.9	94.5	52.1	78.4	
	11	95.1	143	82.8	124	69.2	104	54.1	81.2	45.4	68.2	
	12	79.9	120	69.5	105	58.2	87.5	45.7	68.7	39.0	58.6	
	13	68.1	102	59.3	89.1	49.6	74.6	39.0	58.6	33.2	49.9	
	14	58.7	88.3	51.1	76.8	42.8	64.3	33.6	50.5	28.6	43.1	
	15	51.2	76.9	44.5	66.9	37.3	56.0	29.3	44.0	25.0	37.5	
	16	45.0	67.6	39.1	58.8	32.8	49.2	25.7	38.7	21.9	33.0	
	17							22.8	34.2	19.4	29.2	
Y-Y Axis	0	250	376	212	319	173	259	129	194	101	151	4
	6	237	357	200	301	161	242	106	159	77.7	117	
	8	229	344	193	290	155	233	105	159	77.5	116	
	10	218	328	184	276	148	223	104	157	76.9	116	
	12	206	310	174	261	140	210	101	152	75.8	114	
	14	193	290	162	244	130	196	96.1	144	73.5	110	
	16	178	268	150	225	120	181	89.3	134	69.6	105	
	18	163	246	137	206	110	165	81.7	123	64.5	97.0	
	20	148	222	124	186	99.4	149	73.9	111	58.9	88.6	
	22	133	199	111	167	88.9	134	66.1	99.3	53.2	79.9	
	24	118	177	98.2	148	78.6	118	58.4	87.7	47.4	71.3	
	26	103	155	86.1	129	68.7	103	51.0	76.7	41.8	62.9	
	28	89.7	135	74.5	112	59.4	89.3	44.1	66.3	36.4	54.8	
	30	78.1	117	64.9	97.6	51.8	77.9	38.5	57.9	31.8	47.9	
	32	68.7	103	57.1	85.8	45.6	68.5	33.9	51.0	28.1	42.2	
	34	60.9	91.5	50.6	76.0	40.4	60.7	30.1	45.2	24.9	37.4	
	38	48.7	73.2	40.5	60.9	32.4	48.6	24.1	36.3	20.0	30.1	

Properties of 2 angles—3/8 in. back to back

	3/4	5/8	1/2	3/8	5/16
A_g (in.^{2})	11.6	9.84	8.01	6.10	5.12
r_x (in.)	0.974	0.987	1.00	1.02	1.02
r_y (in.)	2.47	2.45	2.42	2.39	2.38

Properties of single angle

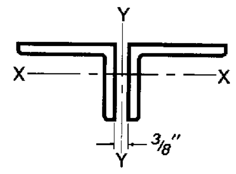
	3/4	5/8	1/2	3/8	5/16
r_z (in.)	0.744	0.746	0.750	0.755	0.758

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L5 SLBB

Shape			2L5×3×									
			1/2		7/16		3/8^c		5/16^c		1/4^c	No. of connectors^a
Wt/ft			25.5		22.5		19.5		16.4		13.2	
Design			P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	
Effective length KL (ft) with respect to indicated axis	X-X Axis	0	162	243	143	215	121	182	94.3	142	67.3	101	b
		1	160	241	141	212	120	180	93.4	140	66.8	100	
		2	155	233	137	205	116	175	90.7	136	65.1	97.8	
		3	146	220	129	194	110	166	86.5	130	62.4	93.8	
		4	135	203	120	180	102	154	80.8	121	58.9	88.5	
		5	122	184	109	163	93.1	140	74.1	111	54.6	82.0	
		6	108	163	96.2	145	82.8	124	66.6	100	49.8	74.8	
		7	93.7	141	83.4	125	72.2	108	58.7	88.3	44.6	67.1	
		8	79.2	119	70.7	106	61.5	92.5	50.8	76.4	39.3	59.1	
		9	65.5	98.5	58.7	88.2	51.4	77.2	43.1	64.8	34.1	51.3	
		10	53.2	80.0	47.8	71.8	42.0	63.1	35.8	53.8	29.1	43.7	
		11	44.0	66.1	39.5	59.3	34.7	52.2	29.6	44.5	24.3	36.6	
		12	37.0	55.6	33.2	49.9	29.2	43.8	24.9	37.4	20.5	30.7	
		13	31.5	47.3	28.3	42.5	24.9	37.4	21.2	31.9	17.4	26.2	
		14							18.3	27.5	15.0	22.6	
	Y-Y Axis	0	162	243	143	215	121	182	94.3	142	67.3	101	4
		6	153	230	134	202	100	150	73.5	110	48.1	72.3	
		8	148	222	130	195	99.8	150	73.3	110	48.1	72.2	
		10	141	212	124	186	99.2	149	73.0	110	47.9	72.0	
		12	134	201	117	176	97.2	146	72.4	109	47.7	71.7	
		14	125	188	110	165	92.7	139	70.8	106	47.3	71.1	
		16	116	175	102	153	86.5	130	67.5	101	46.5	69.9	
		18	107	161	93.7	141	79.7	120	62.9	94.5	44.8	67.3	
		20	97.2	146	85.1	128	72.5	109	57.7	86.8	42.1	63.3	
		22	87.5	132	76.6	115	65.3	98.1	52.4	78.8	38.9	58.5	
		24	78.0	117	68.2	103	58.2	87.4	47.1	70.8	35.5	53.3	
		26	68.9	103	60.2	90.4	51.3	77.1	41.9	63.0	32.0	48.1	
		28	60.0	90.2	52.4	78.7	44.7	67.2	36.9	55.5	28.6	42.9	
		30	52.3	78.6	45.7	68.6	39.0	58.6	32.2	48.4	25.2	37.9	
		32	46.0	69.1	40.2	60.4	34.3	51.5	28.4	42.6	22.3	33.4	
		34	40.8	61.3	35.6	53.5	30.4	45.7	25.2	37.8	19.8	29.7	
		38	32.6	49.1	28.5	42.9	24.4	36.6	20.2	30.3	15.9	23.9	
													5

Properties of 2 angles—3/8 in. back to back

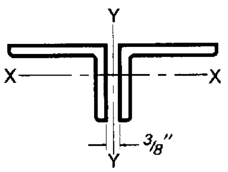
	1/2	7/16	3/8	5/16	1/4
A_g (in.^{2})	7.51	6.62	5.73	4.80	3.88
r_x (in.)	0.824	0.831	0.838	0.846	0.853
r_y (in.)	2.50	2.48	2.47	2.46	2.44

Properties of single angle

	1/2	7/16	3/8	5/16	1/4
r_z (in.)	0.642	0.644	0.646	0.649	0.652

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.



2L4 SLBB

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape			2L4×3$^1/_2$×								No. of connectors[a]
			$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$[c]		
Wt/ft			23.8		18.2		15.3		12.4		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length KL (ft) with respect to indicated axis	X-X Axis	0	151	227	115	173	96.7	145	71.1	107	b
		1	150	225	114	172	96.1	144	70.7	106	
		2	147	221	112	168	94.2	142	69.4	104	
		3	142	213	108	163	91.0	137	67.4	101	
		4	135	203	103	155	86.9	131	64.6	97.0	
		5	127	190	97.0	146	81.7	123	61.1	91.9	
		6	117	176	89.9	135	75.9	114	57.2	85.9	
		7	107	161	82.3	124	69.5	105	52.8	79.4	
		8	96.2	145	74.2	112	62.9	94.5	48.2	72.5	
		9	85.3	128	66.1	99.3	56.1	84.3	43.5	65.4	
		10	74.6	112	58.0	87.2	49.3	74.1	38.8	58.3	
		11	64.3	96.7	50.2	75.5	42.8	64.4	34.1	51.3	
		12	54.6	82.0	42.8	64.4	36.6	55.0	29.7	44.6	
		13	46.5	69.9	36.5	54.8	31.2	46.9	25.5	38.3	
		14	40.1	60.3	31.5	47.3	26.9	40.4	21.9	33.0	
		15	34.9	52.5	27.4	41.2	23.4	35.2	19.1	28.7	
		16	30.7	46.1	24.1	36.2	20.6	31.0	16.8	25.3	
		17	27.2	40.9	21.3	32.1	18.2	27.4	14.9	22.4	
	Y-Y Axis	0	151	227	115	173	96.7	145	71.1	107	3
		6	136	205	101	152	78.2	118	53.4	80.3	
		8	128	192	95.1	143	76.3	115	52.6	79.0	
		10	118	177	87.8	132	72.3	109	50.9	76.5	
		12	107	161	79.6	120	66.2	99.4	47.9	72.0	
		14	95.2	143	70.9	107	59.0	88.6	43.6	65.6	
		16	83.2	125	61.9	93.1	51.4	77.3	38.7	58.1	
		18	71.4	107	53.1	79.8	43.9	66.0	33.6	50.5	
		20	60.2	90.4	44.6	67.1	36.7	55.2	28.6	42.9	
		22	49.9	75.0	37.0	55.6	30.5	45.9	23.9	35.9	
		24	42.0	63.1	31.2	46.9	25.8	38.7	20.2	30.4	
		26	35.8	53.8	26.6	40.0	22.0	33.1	17.3	26.1	
		28	30.9	46.4	23.0	34.5	19.0	28.6	15.0	22.6	
		30	26.9	40.5	20.0	30.1	16.6	25.0	13.1	19.7	

Properties of 2 angles—$^3/_8$ in. back to back

A_g (in.^{2})	7.01	5.34	4.50	3.62	
r_x (in.)	1.04	1.05	1.06	1.07	
r_y (in.)	1.89	1.86	1.85	1.83	

Properties of single angle

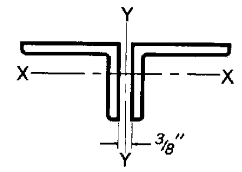
r_z (in.)	0.716	0.719	0.721	0.723	

ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L4 SLBB

Shape			2L4×3×										No. of connectors[a]
			5/8		1/2		3/8		5/16		1/4[c]		
Wt/ft			27.1		22.1		16.9		14.2		11.5		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	168	252	140	211	107	161	89.9	135	66.4	99.8	b
		1	166	249	139	208	106	159	89.0	134	65.8	99.0	
		2	161	242	134	202	103	154	86.4	130	64.1	96.4	
		3	152	229	128	192	97.8	147	82.3	124	61.4	92.2	
		4	142	213	119	179	91.2	137	76.9	116	57.7	86.7	
		5	129	193	108	163	83.4	125	70.4	106	53.3	80.1	
		6	114	172	96.8	145	74.7	112	63.2	95.0	48.4	72.8	
		7	99.7	150	84.6	127	65.7	98.7	55.7	83.7	43.2	64.9	
		8	85.0	128	72.5	109	56.5	85.0	48.1	72.3	37.9	56.9	
		9	71.0	107	60.9	91.5	47.7	71.8	40.7	61.2	32.6	49.0	
		10	58.0	87.2	50.0	75.1	39.4	59.3	33.8	50.8	27.6	41.5	
		11	47.9	72.0	41.3	62.1	32.6	49.0	27.9	42.0	22.9	34.5	
		12	40.3	60.5	34.7	52.2	27.4	41.2	23.5	35.3	19.3	29.0	
		13	34.3	51.6	29.6	44.5	23.3	35.1	20.0	30.0	16.4	24.7	
		14	29.6	44.5	25.5	38.3	20.1	30.2	17.2	25.9	14.2	21.3	
	Y-Y Axis	0	168	252	140	211	107	161	89.9	135	66.4	99.8	3
		6	155	233	128	193	96.3	145	74.1	111	51.1	76.8	
		8	146	220	121	182	90.9	137	72.9	110	50.6	76.0	
		10	136	204	112	169	84.4	127	69.6	105	49.3	74.2	
		12	124	187	103	154	77.1	116	64.1	96.4	46.7	70.2	
		14	112	168	92.3	139	69.2	104	57.7	86.7	42.7	64.2	
		16	99.1	149	81.6	123	61.0	91.7	50.8	76.4	38.1	57.3	
		18	86.4	130	70.9	107	52.9	79.5	44.0	66.1	33.3	50.1	
		20	74.0	111	60.6	91.0	45.1	67.8	37.4	56.1	28.6	43.0	
		22	62.3	93.6	50.8	76.3	37.7	56.7	31.2	46.9	24.1	36.3	
		24	52.4	78.7	42.7	64.2	31.7	47.7	26.3	39.5	20.4	30.6	
		26	44.6	67.1	36.4	54.7	27.1	40.7	22.4	33.7	17.4	26.2	
		28	38.5	57.9	31.4	47.2	23.4	35.1	19.4	29.1	15.1	22.6	
		30	33.5	50.4	27.4	41.1	20.4	30.6	16.9	25.4	13.1	19.8	
		32	29.5	44.3	24.1	36.2	17.9	26.9					4

Properties of 2 angles—3/8 in. back to back

	5/8	1/2	3/8	5/16	1/4
A_g (in.^{2})	7.78	6.50	4.96	4.18	3.38
r_x (in.)	0.845	0.858	0.873	0.880	0.887
r_y (in.)	1.98	1.95	1.93	1.91	1.90

Properties of single angle

	5/8	1/2	3/8	5/16	1/4
r_z (in.)	0.631	0.633	0.636	0.638	0.639

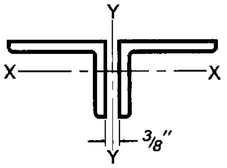
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.

[b] For required number of intermediate connectors, see the discussion of Table 4-8.

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



2L3$^1/_2$ SLBB

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape		2L3$^1/_2$×3×										No. of connectors[a]
		$^1/_2$		$^7/_{16}$		$^3/_8$		$^5/_{16}$		$^1/_4$[c]		
Wt/ft		20.6		18.2		15.8		13.3		10.8		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis — X-X Axis	0	129	194	114	172	99.2	149	83.2	125	64.9	97.6	b
	1	128	192	113	170	98.2	148	82.4	124	64.3	96.7	
	2	124	187	110	165	95.5	143	80.2	120	62.6	94.2	
	3	118	178	105	157	91.0	137	76.5	115	59.9	90.1	
	4	110	166	97.8	147	85.1	128	71.6	108	56.3	84.6	
	5	101	152	89.7	135	78.2	117	65.8	99.0	52.0	78.1	
	6	90.7	136	80.6	121	70.4	106	59.4	89.3	47.1	70.9	
	7	79.8	120	71.1	107	62.2	93.5	52.6	79.1	42.0	63.1	
	8	68.9	103	61.5	92.4	53.9	81.0	45.7	68.7	36.8	55.3	
	9	58.2	87.5	52.1	78.3	45.8	68.9	39.0	58.6	31.6	47.5	
	10	48.2	72.4	43.3	65.1	38.2	57.4	32.6	49.0	26.7	40.1	
	11	39.8	59.9	35.8	53.8	31.6	47.5	27.0	40.5	22.2	33.3	
	12	33.5	50.3	30.1	45.2	26.5	39.9	22.7	34.1	18.6	28.0	
	13	28.5	42.9	25.6	38.5	22.6	34.0	19.3	29.0	15.9	23.9	
	14	24.6	37.0	22.1	33.2	19.5	29.3	16.6	25.0	13.7	20.6	
	15							14.5	21.8	11.9	17.9	
Y-Y Axis	0	129	194	114	172	99.2	149	83.2	125	64.9	97.6	3
	6	115	173	101	152	86.7	130	71.4	107	51.1	76.8	
	8	106	160	93.2	140	80.0	120	66.0	99.2	49.5	74.3	
	10	96.0	144	84.1	126	72.2	109	59.6	89.6	45.9	69.0	
	12	84.7	127	74.2	112	63.7	95.7	52.6	79.1	41.0	61.6	
	14	73.1	110	63.9	96.1	54.8	82.4	45.3	68.1	35.4	53.2	
	16	61.6	92.6	53.8	80.9	46.1	69.3	38.1	57.2	29.8	44.9	
	18	50.8	76.3	44.3	66.5	37.8	56.9	31.2	46.9	24.5	36.8	
	20	41.2	61.9	35.9	54.0	30.7	46.2	25.3	38.1	20.0	30.0	
	22	34.1	51.2	29.7	44.7	25.4	38.2	21.0	31.5	16.6	24.9	
	24	28.7	43.1	25.0	37.6	21.4	32.1	17.7	26.6	14.0	21.0	
	26	24.4	36.7	21.3	32.0	18.2	27.4	15.1	22.7	11.9	18.0	
	28	21.1	31.7									

Properties of 2 angles—$^3/_8$ in. back to back

	$^1/_2$	$^7/_{16}$	$^3/_8$	$^5/_{16}$	$^1/_4$
A_g (in.^{2})	6.00	5.30	4.60	3.86	3.12
r_x (in.)	0.877	0.885	0.892	0.900	0.908
r_y (in.)	1.69	1.67	1.66	1.65	1.63

Properties of single angle

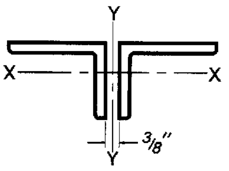
	$^1/_2$	$^7/_{16}$	$^3/_8$	$^5/_{16}$	$^1/_4$
r_z (in.)	0.618	0.620	0.622	0.624	0.628

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L3½ SLBB

Shape		2L3½×2½×								No. of connectors[a]
		½		3/8		5/16		¼[c]		
Wt/ft		18.8		14.5		12.2		9.88		
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length KL (ft) with respect to indicated axis — X-X Axis	0	119	178	91.0	137	76.7	115	59.9	90.1	b
	1	117	175	89.6	135	75.6	114	59.1	88.8	
	2	111	168	85.7	129	72.4	109	56.7	85.3	
	3	103	155	79.6	120	67.4	101	53.0	79.6	
	4	92.7	139	71.8	108	60.9	91.5	48.1	72.3	
	5	80.7	121	62.8	94.4	53.4	80.3	42.5	63.9	
	6	68.1	102	53.4	80.2	45.5	68.4	36.6	55.0	
	7	55.7	83.8	44.0	66.2	37.7	56.7	30.6	46.0	
	8	44.1	66.3	35.2	53.0	30.3	45.6	24.9	37.5	
	9	34.9	52.4	27.8	41.9	24.0	36.1	19.8	29.8	
	10	28.3	42.5	22.6	33.9	19.4	29.2	16.1	24.1	
	11	23.3	35.1	18.6	28.0	16.1	24.1	13.3	19.9	
	12					13.5	20.3	11.2	16.8	
Y-Y Axis	0	119	178	91.0	137	76.7	115	59.9	90.1	4
	2	116	175	88.4	133	73.7	111	48.4	72.7	
	4	113	170	85.7	129	71.4	107	48.3	72.5	
	6	107	161	81.3	122	67.8	102	47.9	72.1	
	8	99.8	150	75.6	114	63.1	94.8	47.0	70.6	
	10	91.0	137	68.9	104	57.4	86.3	44.3	66.5	
	12	81.3	122	61.4	92.3	51.2	76.9	40.0	60.1	
	14	71.1	107	53.6	80.5	44.6	67.1	35.0	52.7	
	16	60.9	91.6	45.8	68.8	38.1	57.3	30.0	45.1	
	18	51.2	76.9	38.3	57.5	31.8	47.8	25.1	37.8	
	20	42.0	63.1	31.3	47.1	26.0	39.1	20.6	30.9	
	22	34.7	52.2	25.9	39.0	21.5	32.4	17.1	25.7	
	24	29.2	43.9	21.8	32.8	18.1	27.2	14.4	21.6	
	26	24.9	37.4	18.6	27.9	15.4	23.2	12.3	18.4	
	28	21.5	32.3	16.0	24.1	13.3	20.0	10.6	15.9	

Properties of 2 angles—3/8 in. back to back

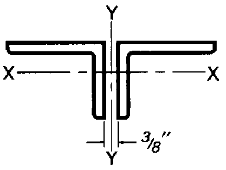
	½	3/8	5/16	¼	
A_g (in.^{2})	5.50	4.22	3.56	2.88	
r_x (in.)	0.701	0.716	0.723	0.731	
r_y (in.)	1.76	1.73	1.72	1.70	

Properties of single angle

	½	3/8	5/16	¼	
r_z (in.)	0.532	0.535	0.538	0.541	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



2L3 SLBB

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape			2L3×2½×												No. of connectors[a]
			½		7/16		3/8		5/16		¼		3/16[c]		
Wt/ft			17.1		15.1		13.1		11.1		8.97		6.82		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	108	162	95.3	143	82.8	124	72.0	108	56.5	84.9	39.1	58.8	b
		1	106	160	93.9	141	81.6	123	71.0	107	55.7	83.7	38.7	58.1	
		2	102	153	89.9	135	78.2	118	68.1	102	53.5	80.4	37.3	56.0	
		3	94.4	142	83.7	126	72.9	110	63.5	95.5	50.0	75.1	35.1	52.7	
		4	85.2	128	75.6	114	66.0	99.2	57.6	86.6	45.4	68.3	32.2	48.4	
		5	74.6	112	66.4	99.8	58.1	87.3	50.9	76.5	40.2	60.4	28.9	43.4	
		6	63.5	95.4	56.6	85.1	49.7	74.7	43.7	65.6	34.6	52.0	25.2	37.9	
		7	52.4	78.8	46.9	70.6	41.3	62.1	36.4	54.8	29.0	43.5	21.5	32.4	
		8	42.0	63.1	37.8	56.8	33.4	50.2	29.6	44.5	23.6	35.5	17.9	27.0	
		9	33.2	49.9	29.9	44.9	26.5	39.8	23.5	35.3	18.8	28.2	14.6	21.9	
		10	26.9	40.4	24.2	36.4	21.4	32.2	19.0	28.6	15.2	22.9	11.8	17.7	
		11	22.2	33.4	20.0	30.1	17.7	26.6	15.7	23.6	12.6	18.9	9.74	14.6	
		12			16.8	25.3	14.9	22.4	13.2	19.9	10.6	15.9	8.18	12.3	
	Y-Y Axis	0	108	162	95.3	143	82.8	124	72.0	108	56.5	84.9	39.1	58.8	3
		2	105	158	92.6	139	79.9	120	68.6	103	52.5	78.9	29.9	44.9	
		4	101	151	88.6	133	76.4	115	65.6	98.6	50.2	75.5	29.7	44.6	
		6	93.5	141	82.2	124	70.9	107	60.9	91.5	46.7	70.2	29.3	44.0	
		8	84.4	127	74.1	111	63.9	96.0	54.9	82.4	42.1	63.3	28.1	42.2	
		10	73.9	111	64.9	97.5	55.8	83.9	47.9	72.0	36.9	55.4	25.6	38.4	
		12	62.9	94.5	55.1	82.8	47.3	71.1	40.6	61.0	31.2	46.9	22.1	33.3	
		14	51.9	78.1	45.4	68.2	38.9	58.5	33.3	50.1	25.6	38.5	18.5	27.7	
		16	41.6	62.5	36.2	54.4	31.0	46.5	26.5	39.8	20.3	30.5	14.9	22.4	
		18	32.9	49.4	28.7	43.1	24.5	36.8	21.0	31.5	16.1	24.2	11.9	17.8	
		20	26.7	40.1	23.2	34.9	19.9	29.9	17.0	25.6	13.1	19.7	9.67	14.5	
		22	22.0	33.1	19.2	28.9	16.4	24.7	14.1	21.1	10.8	16.3	8.03	12.1	
		24	18.5	27.8	16.1	24.3	13.8	20.8	11.8	17.8	9.12	13.7			

Properties of 2 angles—3/8 in. back to back

	½	7/16	3/8	5/16	¼	3/16
A_g (in.^{2})	5.00	4.42	3.84	3.34	2.62	1.99
r_x (in.)	0.718	0.724	0.731	0.739	0.746	0.753
r_y (in.)	1.49	1.48	1.46	1.45	1.44	1.42

Properties of single angle

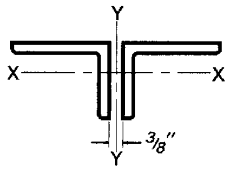
	½	7/16	3/8	5/16	¼	3/16
r_z (in.)	0.516	0.516	0.517	0.518	0.520	0.521

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4-10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB



2L3 SLBB

Shape			2L3×2×										No. of connectors[a]
			1/2		3/8		5/16		1/4		3/16[c]		
Wt/ft			15.4		11.9		10.1		8.18		6.24		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	97.0	146	74.6	112	62.9	94.6	51.3	77.1	35.4	53.3	b
		1	94.5	142	72.8	109	61.5	92.4	50.1	75.3	34.7	52.2	
		2	87.5	132	67.6	102	57.2	86.0	46.7	70.2	32.6	49.0	
		3	77.0	116	59.8	89.8	50.7	76.2	41.6	62.5	29.4	44.2	
		4	64.3	96.6	50.3	75.6	42.9	64.5	35.3	53.0	25.4	38.2	
		5	51.0	76.6	40.3	60.6	34.6	51.9	28.6	43.0	21.1	31.7	
		6	38.4	57.7	30.8	46.3	26.5	39.9	22.1	33.2	16.8	25.2	
		7	28.2	42.5	22.7	34.2	19.7	29.5	16.4	24.7	12.8	19.2	
		8	21.6	32.5	17.4	26.1	15.0	22.6	12.6	18.9	9.79	14.7	
		9	17.1	25.7	13.7	20.7	11.9	17.9	9.9	14.9	7.73	11.6	
	Y-Y Axis	0	97.0	146	74.6	112	62.9	94.6	51.3	77.1	35.4	53.3	4
		2	95.3	143	72.9	110	61.2	91.9	49.2	73.9	27.6	41.5	
		4	91.6	138	70.0	105	58.7	88.3	47.3	71.0	27.5	41.4	
		6	85.8	129	65.5	98.4	54.9	82.5	44.2	66.4	27.3	41.1	
		8	78.3	118	59.6	89.6	49.9	75.0	40.2	60.4	26.7	40.1	
		10	69.6	105	52.8	79.4	44.2	66.4	35.5	53.4	24.7	37.1	
		12	60.3	90.6	45.5	68.4	38.0	57.1	30.5	45.9	21.7	32.6	
		14	50.8	76.4	38.2	57.4	31.8	47.8	25.5	38.4	18.4	27.6	
		16	41.7	62.7	31.2	46.9	25.9	39.0	20.8	31.2	15.2	22.8	
		18	33.4	50.2	24.9	37.4	20.6	31.0	16.5	24.8	12.2	18.3	
		20	27.0	40.6	20.1	30.3	16.7	25.1	13.4	20.1	9.89	14.9	
		22	22.4	33.6	16.7	25.0	13.8	20.8	11.1	16.6	8.19	12.3	
		24	18.8	28.2	14.0	21.0	11.6	17.5	9.30	14.0	6.90	10.4	
		26	16.0	24.1									

Properties of 2 angles—3/8 in. back to back

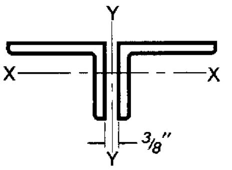
	1/2	3/8	5/16	1/4	3/16	
A_g (in.^{2})	4.50	3.46	2.92	2.38	1.80	
r_x (in.)	0.543	0.555	0.562	0.569	0.577	
r_y (in.)	1.56	1.54	1.52	1.51	1.49	

Properties of single angle

r_z (in.)	0.425	0.426	0.428	0.431	0.435	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with $F_y = 36$ ksi.
Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



2L2½ SLBB

Table 4–10 (continued)
Available Strength in Axial Compression, kips
Double Angles—SLBB

F_y = 36 ksi

Shape			2L2½×2×								No. of connectors[a]
			3/8		5/16		1/4		3/16[c]		
Wt/ft			10.6		8.97		7.30		5.57		
Design			P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	
Effective length *KL* (ft) with respect to indicated axis	X-X Axis	0	66.8	100	56.5	84.9	45.7	68.7	34.3	51.5	
		1	65.3	98.2	55.2	83.0	44.7	67.2	33.6	50.5	
		2	61.0	91.6	51.6	77.6	41.9	62.9	31.5	47.4	
		3	54.3	81.7	46.2	69.4	37.5	56.4	28.4	42.7	
		4	46.3	69.5	39.5	59.3	32.2	48.4	24.5	36.9	
		5	37.6	56.5	32.2	48.5	26.5	39.8	20.3	30.6	
		6	29.2	43.9	25.2	37.9	20.8	31.3	16.2	24.3	b
		7	21.8	32.7	18.9	28.4	15.7	23.6	12.3	18.5	
		8	16.7	25.1	14.4	21.7	12.0	18.0	9.40	14.1	
		9	13.2	19.8	11.4	17.2	9.48	14.2	7.43	11.2	
	Y-Y Axis	0	66.8	100	56.5	84.9	45.7	68.7	34.3	51.5	
		2	64.8	97.4	54.3	81.7	43.4	65.2	27.7	41.6	
		4	61.1	91.8	51.1	76.8	40.8	61.3	27.4	41.3	
		6	55.4	83.2	46.2	69.4	36.8	55.4	26.4	39.7	
		8	48.2	72.5	40.0	60.1	31.9	48.0	23.6	35.4	
		10	40.4	60.7	33.2	50.0	26.5	39.8	19.7	29.5	3
		12	32.5	48.9	26.5	39.8	21.1	31.7	15.6	23.5	
		14	25.1	37.7	20.2	30.4	16.0	24.1	11.9	17.9	
		16	19.2	28.9	15.5	23.3	12.3	18.5	9.15	13.8	
		18	15.2	22.9	12.3	18.4	9.74	14.6	7.26	10.9	
		20	12.3	18.5	9.94	14.9	7.90	11.9	5.89	8.86	
											4

Properties of 2 angles—3/8 in. back to back

	3/8	5/16	1/4	3/16
A_g (in.^{2})	3.10	2.62	2.12	1.62
r_x (in.)	0.574	0.581	0.589	0.597
r_y (in.)	1.27	1.26	1.24	1.23

Properties of single angle

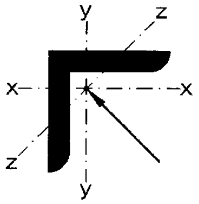
	3/8	5/16	1/4	3/16
r_z (in.)	0.419	0.420	0.423	0.426

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[a] For Y-Y axis, welded or pretensioned bolted intermediate connectors must be used.
[b] For required number of intermediate connectors, see the discussion of Table 4-8.
[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–11
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

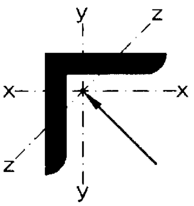


L8

Shape		L8×8×											
		1$^1/_8$		1		$^7/_8$		$^3/_4$		$^5/_8$		$^9/_{16}{}^c$	
Wt/ft		57.2		51.3		45.3		39.2		33.0		29.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	360	541	323	486	285	428	246	369	207	311	179	270
	1	359	539	322	484	284	426	245	368	206	310	179	269
	2	356	534	319	480	281	422	243	365	204	307	177	267
	3	350	526	314	473	277	416	239	359	201	302	175	263
	4	342	515	308	462	271	407	234	352	197	296	171	258
	5	333	500	299	450	263	396	228	342	192	288	167	251
	6	322	484	289	435	255	383	220	331	185	278	162	243
	7	309	464	278	417	245	368	211	318	178	268	156	234
	8	295	443	265	398	234	351	202	304	170	256	149	224
	9	280	420	251	378	222	333	192	288	162	243	142	213
	10	264	396	237	356	209	314	181	272	153	229	134	202
	11	247	371	222	334	196	294	170	255	143	215	126	190
	12	230	345	207	311	182	274	158	238	134	201	118	178
	13	212	319	191	288	169	254	146	220	124	186	110	165
	14	195	294	176	264	155	234	135	203	114	171	102	153
	15	178	268	161	242	142	214	123	185	104	157	93.4	140
	16	162	243	146	219	129	194	112	169	95.1	143	85.4	128
	17	146	220	132	198	117	175	101	152	86.0	129	77.6	117
	18	131	196	118	177	104	157	90.8	137	77.2	116	70.1	105
	19	117	176	106	159	93.7	141	81.5	123	69.3	104	62.9	94.5
	20	106	159	95.5	144	84.6	127	73.6	111	62.5	94.0	56.8	85.3
	21	96.1	144	86.7	130	76.7	115	66.7	100	56.7	85.3	51.5	77.4
	22	87.5	132	79.0	119	69.9	105	60.8	91.4	51.7	77.7	46.9	70.5
	23	80.1	120	72.2	109	64.0	96.1	55.6	83.6	47.3	71.1	42.9	64.5
	24	73.5	111	66.4	99.7	58.7	88.3	51.1	76.8	43.4	65.3	39.4	59.2
	25	67.8	102	61.2	91.9	54.1	81.4	47.1	70.8	40.0	60.2	36.3	54.6
	26	62.7	94.2	56.5	85.0	50.0	75.2	43.5	65.4	37.0	55.6	33.6	50.5
		Properties											
A_g (in.^{2})		16.7		15.0		13.2		11.4		9.61		8.68	
r_z (in.)		1.56		1.56		1.57		1.57		1.58		1.58	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.



L8

Table 4–11 (continued)

Available Strength in Axial Compression, kips

Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L8×8×		L8×6×									
		$^1/_2$^c^		1		$^7/_8$		$^3/_4$		$^5/_8$		$^9/_{16}$^c^	
Wt/ft		26.7		44.4		39.3		34.0		28.6		25.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	152	229	280	421	248	373	214	322	180	270	156	235
	1	152	228	279	419	247	371	213	321	179	269	156	234
	2	151	226	275	413	243	366	210	316	176	265	154	231
	3	149	223	269	404	238	357	206	309	173	259	150	226
	4	146	219	260	391	230	346	199	299	167	251	146	219
	5	142	214	249	375	221	332	191	287	161	241	140	211
	6	138	207	237	356	210	316	182	273	153	230	134	201
	7	133	200	223	335	198	297	171	257	144	216	126	190
	8	128	192	208	313	184	277	160	240	135	202	119	178
	9	122	183	192	289	171	256	148	222	125	187	110	166
	10	116	174	176	265	156	235	136	204	114	172	101	152
	11	109	164	160	240	142	213	123	185	104	156	92.6	139
	12	103	154	143	216	127	192	111	167	93.7	141	83.9	126
	13	95.8	144	128	192	114	171	98.8	148	83.7	126	75.3	113
	14	88.9	134	113	169	100	151	87.3	131	74.1	111	67.0	101
	15	82.1	123	98.3	148	87.5	132	76.3	115	64.8	97.5	59.0	88.6
	16	75.4	113	86.4	130	76.9	116	67.1	101	57.0	85.6	51.8	77.9
	17	68.9	104	76.6	115	68.2	102	59.4	89.3	50.5	75.9	45.9	69.0
	18	62.6	94.1	68.3	103	60.8	91.4	53.0	79.6	45.0	67.7	41.0	61.6
	19	56.4	84.8	61.3	92.1	54.6	82.0	47.6	71.5	40.4	60.7	36.8	55.2
	20	50.9	76.5	55.3	83.1	49.2	74.0	42.9	64.5	36.5	54.8	33.2	49.9
	21	46.2	69.4	50.2	75.4	44.7	67.1	38.9	58.5	33.1	49.7	30.1	45.2
	22	42.1	63.3										
	23	38.5	57.9										
	24	35.4	53.2										
	25	32.6	49.0										
	26	30.1	45.3										

Properties

	L8×8×$^1/_2$	L8×6×1	L8×6×$^7/_8$	L8×6×$^3/_4$	L8×6×$^5/_8$	L8×6×$^9/_{16}$
A_g (in.^{2})	7.75	13.0	11.5	9.94	8.36	7.56
r_z (in.)	1.59	1.28	1.28	1.29	1.29	1.30

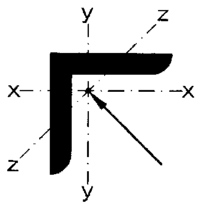
ASD	LRFD
Ω_c = 1.67	ϕ_c = 0.90

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L8

Shape		L8×6×				L8×4×							
		$^1/_2$^{c}		$^7/_{16}$^{c}		1		$^7/_8$		$^3/_4$		$^5/_8$	
Wt/ft		23.2		20.4		37.6		33.3		28.9		24.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	**0**	133	199	109	163	237	356	210	315	182	273	153	230
	1	132	199	108	163	235	353	208	312	180	271	151	227
	2	130	196	107	161	227	342	201	302	174	262	147	220
	3	128	192	105	158	215	324	191	287	166	249	139	209
	4	124	187	102	154	200	301	177	266	154	231	130	195
	5	120	180	98.9	149	182	273	161	242	140	210	118	177
	6	115	172	94.9	143	162	243	143	215	125	187	105	158
	7	109	163	90.3	136	141	212	125	188	109	164	92.1	139
	8	102	154	85.3	128	120	180	106	160	93.0	140	78.9	119
	9	95.3	143	80.0	120	100	150	88.9	134	77.8	117	66.2	99.5
	10	88.2	133	74.5	112	81.8	123	72.7	109	63.6	95.7	54.3	81.7
	11	80.9	122	68.8	103	67.6	102	60.1	90.3	52.6	79.1	44.9	67.5
	12	73.7	111	63.0	94.7	56.8	85.3	50.5	75.9	44.2	66.4	37.7	56.7
	13	66.5	100	57.3	86.2	48.4	72.7	43.0	64.6	37.7	56.6	32.1	48.3
	14	59.6	89.5	51.8	77.8	41.7	62.7	37.1	55.7	32.5	48.8	27.7	41.7
	15	52.9	79.5	46.4	69.7								
	16	46.6	70.0	41.2	61.9								
	17	41.3	62.0	36.5	54.8								
	18	36.8	55.3	32.5	48.9								
	19	33.0	49.6	29.2	43.9								
	20	29.8	44.8	26.4	39.6								
	21	27.0	40.6	23.9	35.9								

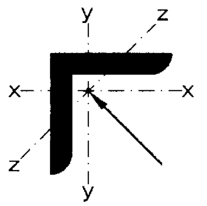
Properties

	L8×6×$^1/_2$	L8×6×$^7/_{16}$	L8×4×1	L8×4×$^7/_8$	L8×4×$^3/_4$	L8×4×$^5/_8$
A_g (in.^{2})	6.75	5.93	11.0	9.73	8.44	7.11
r_z (in.)	1.30	1.31	0.844	0.846	0.850	0.856

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



L8–L7

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L8×4×						L7×4×					
		$^9/_{16}$ ^{c}		$^1/_2$ ^{c}		$^7/_{16}$ ^{c}		$^3/_4$		$^5/_8$		$^1/_2$ ^{c}	
Wt/ft		22.1		19.7		17.4		26.2		22.1		17.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	133	200	113	170	92.8	139	166	249	140	210	109	164
	1	132	198	112	168	92.0	138	164	247	138	208	108	163
	2	128	192	109	164	89.6	135	159	239	134	202	105	158
	3	122	183	104	156	85.9	129	151	227	127	191	100	150
	4	114	171	97.4	146	80.9	122	140	211	119	178	93.4	140
	5	104	156	89.6	135	74.9	113	128	192	108	162	85.6	129
	6	93.2	140	80.9	122	68.1	102	114	172	96.6	145	76.9	116
	7	82.0	123	71.7	108	60.9	91.6	99.8	150	84.5	127	67.7	102
	8	70.8	106	62.4	93.8	53.6	80.5	85.4	128	72.5	109	58.5	87.9
	9	59.9	90.0	53.3	80.1	46.3	69.6	71.6	108	60.9	91.5	49.5	74.5
	10	49.5	74.4	44.7	67.1	39.4	59.2	58.7	88.3	50.0	75.2	41.1	61.8
	11	40.9	61.5	36.9	55.5	32.8	49.3	48.5	73.0	41.3	62.1	34.0	51.0
	12	34.4	51.7	31.0	46.6	27.6	41.5	40.8	61.3	34.7	52.2	28.5	42.9
	13	29.3	44.1	26.4	39.7	23.5	35.3	34.8	52.2	29.6	44.5	24.3	36.5
	14	25.3	38.0	22.8	34.3	20.3	30.5	30.0	45.0	25.5	38.4	21.0	31.5

Properties

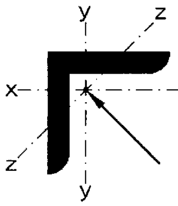
A_g (in.^{2})	6.43	5.75	5.06	7.69	6.48	5.25
r_z (in.)	0.859	0.863	0.867	0.855	0.860	0.866

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L7–L6

Shape		L7×4×				L6×6×							
		$^7/_{16}{}^c$		$^3/_8{}^c$		1		$^7/_8$		$^3/_4$		$^5/_8$	
Wt/ft		15.8		13.6		37.5		33.2		28.8		24.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	**0**	**90.8**	136	**72.1**	108	**238**	357	**210**	316	**182**	274	**154**	231
	1	**90.0**	135	**71.5**	107	**236**	355	**209**	314	**181**	273	**153**	230
	2	**87.5**	132	**69.7**	105	**232**	349	**206**	309	**178**	268	**150**	226
	3	**83.6**	126	**66.9**	100	**226**	340	**200**	301	**173**	261	**146**	220
	4	**78.4**	118	**63.1**	94.8	**217**	327	**192**	289	**167**	251	**141**	212
	5	**72.2**	109	**58.5**	87.9	**207**	311	**183**	275	**159**	239	**134**	201
	6	**65.3**	98.2	**53.4**	80.2	**194**	292	**172**	259	**149**	224	**126**	190
	7	**58.0**	87.2	**47.9**	71.9	**181**	272	**160**	241	**139**	209	**117**	176
	8	**50.6**	76.0	**42.2**	63.5	**166**	250	**147**	221	**128**	192	**108**	162
	9	**43.3**	65.1	**36.6**	55.1	**151**	227	**134**	201	**116**	175	**98.5**	148
	10	**36.4**	54.7	**31.2**	47.0	**136**	204	**121**	181	**105**	157	**88.7**	133
	11	**30.1**	45.3	**26.2**	39.3	**121**	182	**107**	161	**93.3**	140	**79.0**	119
	12	**25.3**	38.0	**22.0**	33.0	**106**	160	**94.3**	142	**82.1**	123	**69.7**	105
	13	**21.6**	32.4	**18.7**	28.2	**92.4**	139	**82.0**	123	**71.5**	107	**60.7**	91.2
	14	**18.6**	27.9	**16.2**	24.3	**79.7**	120	**70.7**	106	**61.7**	92.7	**52.4**	78.7
	15					**69.4**	104	**61.6**	92.6	**53.7**	80.7	**45.6**	68.6
	16					**61.0**	91.7	**54.2**	81.4	**47.2**	71.0	**40.1**	60.3
	17					**54.0**	81.2	**48.0**	72.1	**41.8**	62.8	**35.5**	53.4
	18					**48.2**	72.5	**42.8**	64.3	**37.3**	56.1	**31.7**	47.6
	19					**43.3**	65.0	**38.4**	57.7	**33.5**	50.3	**28.4**	42.7

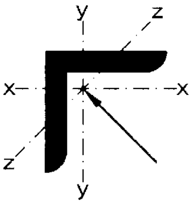
Properties

	L7×4×$^7/_{16}{}^c$	L7×4×$^3/_8{}^c$	L6×6×1	L6×6×$^7/_8$	L6×6×$^3/_4$	L6×6×$^5/_8$
A_g (in.^{2})	4.62	3.98	11.0	9.75	8.46	7.13
r_z (in.)	0.869	0.873	1.17	1.17	1.17	1.17

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



L6

Table 4–11 (continued)
Available Strength in Axial Compression, kips

F_y = 36 ksi

Concentrically Loaded Single Angles

Shape		L6×6×										L6×4×	
		$^{9}/_{16}$		$^{1}/_{2}$		$^{7}/_{16}{}^{c}$		$^{3}/_{8}{}^{c}$		$^{5}/_{16}{}^{c}$		$^{7}/_{8}$	
Wt/ft		22.0		19.6		17.3		14.9		12.5		27.2	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	139	209	124	187	106	160	86.1	129	65.4	98.3	172	259
	1	138	208	124	186	106	159	85.6	129	65.1	97.8	170	256
	2	136	205	122	183	104	157	84.4	127	64.2	96.5	165	248
	3	132	199	118	178	102	153	82.3	124	62.8	94.4	157	236
	4	127	192	114	171	97.9	147	79.6	120	60.9	91.6	146	219
	5	121	182	109	163	93.4	140	76.1	114	58.5	88.0	133	199
	6	114	172	102	154	88.1	132	72.1	108	55.8	83.8	118	178
	7	106	160	95.3	143	82.3	124	67.7	102	52.6	79.1	103	155
	8	98.0	147	87.8	132	76.0	114	62.9	94.5	49.3	74.1	88.5	133
	9	89.3	134	80.0	120	69.5	104	57.8	86.9	45.7	68.7	74.1	111
	10	80.5	121	72.2	108	62.9	94.5	52.7	79.2	42.0	63.2	60.8	91.3
	11	71.8	108	64.4	96.8	56.3	84.6	47.5	71.4	38.3	57.6	50.2	75.5
	12	63.3	95.1	56.8	85.4	49.9	75.0	42.5	63.8	34.6	52.0	42.2	63.4
	13	55.2	82.9	49.6	74.5	43.7	65.7	37.6	56.4	31.0	46.6	35.9	54.0
	14	47.6	71.6	42.8	64.3	37.9	56.9	32.8	49.4	27.5	41.3	31.0	46.6
	15	41.5	62.4	37.3	56.0	33.0	49.6	28.6	43.0	24.1	36.3		
	16	36.5	54.8	32.8	49.2	29.0	43.6	25.1	37.8	21.2	31.9		
	17	32.3	48.5	29.0	43.6	25.7	38.6	22.3	33.5	18.8	28.2		
	18	28.8	43.3	25.9	38.9	22.9	34.4	19.9	29.9	16.8	25.2		
	19	25.9	38.9	23.2	34.9	20.6	30.9	17.8	26.8	15.0	22.6		

Properties

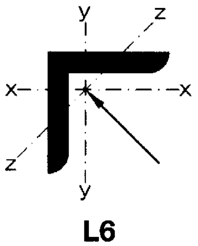
A_g (in.^{2})	6.45	5.77	5.08	4.38	3.67	7.98
r_z (in.)	1.18	1.18	1.18	1.19	1.19	0.854

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



Shape		L6×4×									
		3/4		5/8		9/16		1/2		7/16^c	
Wt/ft		23.6		19.9		18.1		16.2		14.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	**150**	225	**126**	190	**114**	172	**102**	154	**87.7**	132
	1	**148**	222	**125**	188	**113**	170	**101**	152	**86.8**	131
	2	**143**	216	**121**	182	**110**	165	**98.3**	148	**84.3**	127
	3	**136**	205	**115**	173	**104**	157	**93.4**	140	**80.3**	121
	4	**127**	190	**107**	161	**97.2**	146	**87.0**	131	**75.0**	113
	5	**115**	173	**97.7**	147	**88.6**	133	**79.4**	119	**68.6**	103
	6	**103**	155	**87.3**	131	**79.2**	119	**71.0**	107	**61.6**	92.6
	7	**90.0**	135	**76.3**	115	**69.4**	104	**62.2**	93.6	**54.2**	81.5
	8	**77.1**	116	**65.4**	98.4	**59.5**	89.4	**53.5**	80.3	**46.8**	70.4
	9	**64.6**	97.2	**55.0**	82.6	**50.0**	75.2	**45.0**	67.6	**39.6**	59.6
	10	**53.0**	79.7	**45.1**	67.8	**41.1**	61.8	**37.0**	55.6	**32.8**	49.3
	11	**43.8**	65.8	**37.3**	56.1	**34.0**	51.1	**30.6**	46.0	**27.1**	40.8
	12	**36.8**	55.3	**31.3**	47.1	**28.6**	42.9	**25.7**	38.6	**22.8**	34.3
	13	**31.4**	47.1	**26.7**	40.1	**24.3**	36.6	**21.9**	32.9	**19.4**	29.2
	14	**27.0**	40.7	**23.0**	34.6	**21.0**	31.5	**18.9**	28.4	**16.7**	25.2

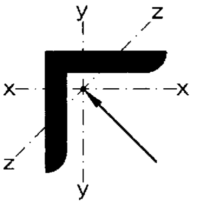
Properties

	3/4	5/8	9/16	1/2	7/16^c
A_g (in.^{2})	6.94	5.86	5.31	4.75	4.18
r_z (in.)	0.856	0.859	0.861	0.864	0.867

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



L6

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape	**L6×4×**				**L6×3½×**					
	3/8ᶜ		**5/16ᶜ**		**1/2**		**3/8ᶜ**		**5/16ᶜ**	
Wt/ft	**12.3**		**10.3**		**15.4**		**11.7**		**9.8**	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z **0**	**70.9**	107	**53.9**	81.0	**97.0**	146	**67.2**	101	**51.1**	76.8
1	**70.3**	106	**53.4**	80.3	**95.7**	144	**66.4**	99.8	**50.6**	76.0
2	**68.4**	103	**52.1**	78.4	**92.0**	138	**64.1**	96.3	**49.0**	73.6
3	**65.3**	98.2	**50.0**	75.2	**86.1**	129	**60.4**	90.8	**46.4**	69.8
4	**61.3**	92.1	**47.2**	71.0	**78.5**	118	**55.6**	83.5	**43.1**	64.8
5	**56.4**	84.8	**43.9**	66.0	**69.6**	105	**50.0**	75.1	**39.2**	58.9
6	**51.1**	76.7	**40.1**	60.3	**60.2**	90.5	**43.8**	65.9	**34.8**	52.4
7	**45.3**	68.2	**36.0**	54.2	**50.7**	76.1	**37.6**	56.5	**30.3**	45.6
8	**39.5**	59.4	**31.9**	47.9	**41.5**	62.4	**31.4**	47.3	**25.9**	38.9
9	**33.9**	50.9	**27.7**	41.7	**33.2**	49.8	**25.7**	38.6	**21.6**	32.4
10	**28.5**	42.8	**23.7**	35.7	**26.9**	40.4	**20.8**	31.2	**17.6**	26.5
11	**23.6**	35.4	**19.9**	29.9	**22.2**	33.4	**17.2**	25.8	**14.6**	21.9
12	**19.8**	29.8	**16.7**	25.2	**18.7**	28.0	**14.4**	21.7	**12.2**	18.4
13	**16.9**	25.4	**14.3**	21.4						
14	**14.6**	21.9	**12.3**	18.5						

Properties

	L6×4×3/8	L6×4×5/16	L6×3½×1/2	L6×3½×3/8	L6×3½×5/16
A_g (in.²)	3.61	3.03	4.50	3.42	2.87
r_z (in.)	0.870	0.874	0.756	0.763	0.767

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

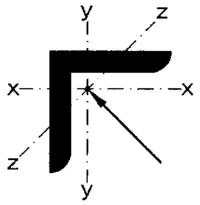
ᶜ Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4-11 (continued)

Available Strength in Axial Compression, kips

Concentrically Loaded Single Angles



L5

Shape		L5×5×											
		7/8		3/4		5/8		1/2		7/16		3/8[c]	
Wt/ft		27.3		23.7		20.1		16.3		14.4		12.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	172	259	150	225	126	190	102	154	90.1	135	76.5	115
	1	171	256	148	223	125	188	102	153	89.4	134	75.9	114
	2	167	250	145	218	122	184	99.2	149	87.3	131	74.2	112
	3	160	240	139	209	118	177	95.4	143	84.0	126	71.4	107
	4	151	227	132	198	111	167	90.2	136	79.5	119	67.7	102
	5	141	211	122	184	104	156	84.1	126	74.1	111	63.2	94.9
	6	129	194	112	168	94.8	143	77.1	116	67.9	102	58.1	87.3
	7	116	174	101	152	85.5	128	69.6	105	61.4	92.2	52.6	79.0
	8	103	155	89.5	135	75.8	114	61.8	92.9	54.5	82.0	46.9	70.4
	9	89.7	135	78.1	117	66.2	99.6	54.0	81.2	47.7	71.7	41.1	61.8
	10	77.0	116	67.1	101	56.9	85.6	46.5	69.9	41.1	61.8	35.6	53.5
	11	64.9	97.5	56.6	85.0	48.1	72.3	39.4	59.2	34.9	52.4	30.3	45.5
	12	54.5	81.9	47.5	71.5	40.4	60.7	33.1	49.7	29.3	44.0	25.5	38.3
	13	46.4	69.8	40.5	60.9	34.4	51.7	28.2	42.4	25.0	37.5	21.7	32.6
	14	40.0	60.2	34.9	52.5	29.7	44.6	24.3	36.5	21.5	32.3	18.7	28.1
	15	34.9	52.4	30.4	45.7	25.9	38.9	21.2	31.8	18.7	28.2	16.3	24.5
	16	30.7	46.1	26.7	40.2	22.7	34.2	18.6	28.0	16.5	24.8	14.3	21.5

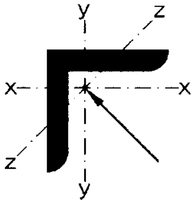
Properties

	7/8	3/4	5/8	1/2	7/16	3/8[c]
A_g (in.^{2})	7.98	6.94	5.86	4.75	4.18	3.61
r_z (in.)	0.971	0.972	0.975	0.980	0.983	0.986

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



L5

Table 4–11 (continued)

Available Strength in Axial Compression, kips

Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L5×5×		L5×3½×									
		5/16^c		3/4		5/8		1/2		3/8^c		5/16^c	
Wt/ft		10.4		19.8		16.8		13.6		10.4		8.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	**0**	**59.5**	89.5	**125**	188	**106**	159	**86.3**	130	**64.6**	97.1	**50.3**	75.7
	1	**59.1**	88.9	**124**	186	**105**	157	**85.1**	128	**63.8**	95.9	**49.7**	74.8
	2	**57.9**	87.0	**119**	178	**100**	151	**81.8**	123	**61.3**	92.2	**48.0**	72.1
	3	**55.9**	84.0	**111**	166	**93.8**	141	**76.4**	115	**57.5**	86.4	**45.2**	67.9
	4	**53.2**	79.9	**101**	151	**85.3**	128	**69.5**	105	**52.4**	78.8	**41.5**	62.4
	5	**49.9**	75.0	**89.0**	134	**75.5**	113	**61.6**	92.6	**46.6**	70.0	**37.3**	56.0
	6	**46.2**	69.4	**76.5**	115	**65.0**	97.6	**53.1**	79.8	**40.4**	60.7	**32.6**	49.1
	7	**42.2**	63.4	**64.1**	96.3	**54.4**	81.8	**44.6**	67.0	**34.0**	51.2	**27.9**	42.0
	8	**37.9**	57.0	**52.2**	78.4	**44.4**	66.7	**36.4**	54.7	**28.0**	42.1	**23.3**	35.0
	9	**33.6**	50.6	**41.5**	62.4	**35.3**	53.0	**29.0**	43.6	**22.4**	33.7	**19.0**	28.5
	10	**29.4**	44.2	**33.6**	50.5	**28.6**	43.0	**23.5**	35.3	**18.1**	27.3	**15.4**	23.1
	11	**25.4**	38.1	**27.8**	41.7	**23.6**	35.5	**19.4**	29.2	**15.0**	22.5	**12.7**	19.1
	12	**21.5**	32.4	**23.3**	35.1	**19.9**	29.8	**16.3**	24.5	**12.6**	18.9	**10.7**	16.0
	13	**18.3**	27.6										
	14	**15.8**	23.8										
	15	**13.8**	20.7										
	16	**12.1**	18.2										

Properties

	L5×5×5/16^c	L5×3½×3/4	5/8	1/2	3/8^c	5/16^c
A_g (in.^{2})	3.03	5.81	4.92	4.00	3.05	2.56
r_z (in.)	0.990	0.744	0.746	0.750	0.755	0.758

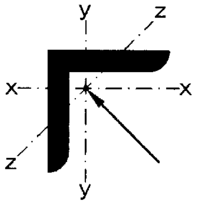
ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L5

Shape		L5×3½×		L5×3×									
		1/4^c		1/2		7/16		3/8^c		5/16^c		1/4^c	
Wt/ft		7.0		12.8		11.3		9.7		8.2		6.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	35.7	53.7	80.9	122	71.4	107	60.7	91.2	47.2	70.9	33.7	50.6
	1	35.3	53.1	79.4	119	70.1	105	59.6	89.6	46.4	69.7	33.2	49.9
	2	34.2	51.5	75.2	113	66.4	99.7	56.5	84.9	44.2	66.4	31.8	47.8
	3	32.5	48.8	68.6	103	60.6	91.0	51.7	77.7	40.7	61.2	29.6	44.5
	4	30.2	45.4	60.3	90.6	53.3	80.1	45.6	68.5	36.3	54.5	26.8	40.2
	5	27.4	41.3	51.1	76.8	45.2	67.9	38.8	58.4	31.3	47.0	23.5	35.3
	6	24.4	36.7	41.7	62.7	37.0	55.6	31.9	48.0	26.1	39.3	20.1	30.2
	7	21.3	32.0	32.8	49.4	29.2	43.8	25.3	38.1	21.1	31.7	16.7	25.0
	8	18.2	27.3	25.2	37.9	22.4	33.7	19.5	29.3	16.5	24.8	13.4	20.2
	9	15.2	22.9	19.9	29.9	17.7	26.6	15.4	23.2	13.0	19.6	10.6	16.0
	10	12.4	18.7	16.1	24.3	14.3	21.5	12.5	18.8	10.6	15.9	8.61	12.9
	11	10.3	15.4										
	12	8.64	13.0										

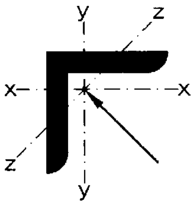
Properties

	5×3½×1/4	5×3×1/2	5×3×7/16	5×3×3/8	5×3×5/16	5×3×1/4
A_g (in.^{2})	2.06	3.75	3.31	2.86	2.40	1.94
r_z (in.)	0.761	0.642	0.644	0.646	0.649	0.652

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



L4

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L4×4×											
		3/4		5/8		1/2		7/16		3/8		5/16	
Wt/ft		18.5		15.7		12.7		11.2		9.7		8.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	117	176	99.3	149	80.8	121	71.4	107	61.7	92.7	51.6	77.5
	1	116	174	98.0	147	79.7	120	70.5	106	60.9	91.5	51.0	76.6
	2	111	168	94.4	142	76.8	115	67.9	102	58.6	88.1	49.1	73.8
	3	105	157	88.6	133	72.1	108	63.7	95.8	55.1	82.8	46.1	69.4
	4	95.8	144	81.1	122	66.0	99.2	58.4	87.7	50.5	75.9	42.3	63.6
	5	85.4	128	72.3	109	58.9	88.6	52.1	78.4	45.1	67.8	37.8	56.9
	6	74.3	112	62.9	94.6	51.3	77.1	45.4	68.3	39.3	59.1	33.0	49.6
	7	63.1	94.8	53.4	80.2	43.6	65.5	38.6	58.0	33.4	50.2	28.1	42.2
	8	52.1	78.4	44.1	66.4	36.0	54.2	32.0	48.0	27.7	41.7	23.3	35.1
	9	42.0	63.1	35.5	53.4	29.0	43.6	25.8	38.7	22.4	33.6	18.9	28.3
	10	34.0	51.1	28.8	43.3	23.5	35.4	20.9	31.4	18.1	27.2	15.3	23.0
	11	28.1	42.2	23.8	35.7	19.4	29.2	17.2	25.9	15.0	22.5	12.6	19.0
	12	23.6	35.5	20.0	30.0	16.3	24.6	14.5	21.8	12.6	18.9	10.6	15.9
	13											9.04	13.6

Properties

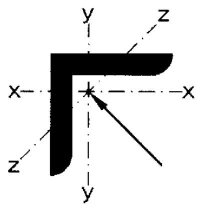
	3/4	5/8	1/2	7/16	3/8	5/16
A_g (in.^{2})	5.44	4.61	3.75	3.31	2.86	2.40
r_z (in.)	0.774	0.774	0.776	0.777	0.779	0.781

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L4

Shape		L4×4×		L4×3½×								L4×3×	
		1/4^c		1/2		3/8		5/16		1/4^c		5/8	
Wt/ft		6.6		11.9		9.1		7.7		6.2		13.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	38.1	57.3	75.5	114	57.6	86.5	48.4	72.7	35.6	53.5	83.9	126
	1	37.7	56.7	74.4	112	56.7	85.2	47.7	71.7	35.1	52.8	82.3	124
	2	36.4	54.8	71.2	107	54.3	81.6	45.6	68.6	33.7	50.7	77.7	117
	3	34.4	51.8	66.1	99.4	50.4	75.8	42.4	63.8	31.6	47.5	70.7	106
	4	31.8	47.8	59.6	89.6	45.5	68.4	38.3	57.6	28.8	43.3	61.9	93.0
	5	28.8	43.2	52.2	78.4	39.9	60.0	33.6	50.6	25.6	38.4	52.1	78.4
	6	25.4	38.2	44.3	66.6	34.0	51.0	28.7	43.1	22.1	33.2	42.3	63.6
	7	21.9	33.0	36.6	55.0	28.1	42.2	23.7	35.7	18.6	28.0	33.0	49.6
	8	18.5	27.8	29.3	44.0	22.5	33.9	19.1	28.7	15.3	23.0	25.3	38.0
	9	15.3	23.0	23.1	34.8	17.8	26.8	15.1	22.7	12.2	18.3	20.0	30.0
	10	12.4	18.6	18.7	28.2	14.4	21.7	12.2	18.4	9.89	14.9	16.2	24.3
	11	10.2	15.4	15.5	23.3	11.9	17.9	10.1	15.2	8.17	12.3		
	12	8.61	12.9					8.49	12.8	6.87	10.3		
	13	7.34	11.0										

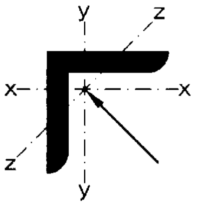
Properties

	L4×4× 1/4^c	L4×3½× 1/2	L4×3½× 3/8	L4×3½× 5/16	L4×3½× 1/4^c	L4×3× 5/8
A_g (in.^{2})	1.94	3.50	2.67	2.25	1.81	3.89
r_z (in.)	0.783	0.716	0.719	0.721	0.723	0.631

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



L4–L3$^{1}/_{2}$

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L4×3×								L3$^{1}/_{2}$×3$^{1}/_{2}$×			
		$^{1}/_{2}$		$^{3}/_{8}$		$^{5}/_{16}$		$^{1}/_{4}{}^{c}$		$^{1}/_{2}$		$^{7}/_{16}$	
Wt/ft		11.1		8.5		7.1		5.8		11.1		9.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	**0**	**70.1**	105	**53.5**	80.4	**44.9**	67.5	**33.2**	49.9	**70.1**	105	**61.9**	93.0
	1	**68.7**	103	**52.5**	78.9	**44.1**	66.3	**32.7**	49.1	**68.9**	104	**60.9**	91.5
	2	**64.9**	97.6	**49.6**	74.5	**41.7**	62.7	**31.0**	46.7	**65.6**	98.6	**57.9**	87.1
	3	**59.1**	88.8	**45.2**	67.9	**38.0**	57.1	**28.5**	42.9	**60.4**	90.8	**53.4**	80.3
	4	**51.7**	77.8	**39.6**	59.5	**33.4**	50.1	**25.3**	38.1	**53.9**	81.0	**47.6**	71.6
	5	**43.6**	65.6	**33.4**	50.3	**28.2**	42.4	**21.8**	32.7	**46.5**	69.8	**41.1**	61.8
	6	**35.4**	53.2	**27.2**	40.9	**23.0**	34.6	**18.1**	27.2	**38.8**	58.3	**34.3**	51.6
	7	**27.7**	41.6	**21.3**	32.0	**18.1**	27.1	**14.5**	21.8	**31.3**	47.1	**27.8**	41.7
	8	**21.2**	31.9	**16.3**	24.6	**13.9**	20.8	**11.3**	16.9	**24.5**	36.8	**21.7**	32.6
	9	**16.8**	25.2	**12.9**	19.4	**10.9**	16.5	**8.91**	13.4	**19.3**	29.0	**17.1**	25.8
	10	**13.6**	20.4	**10.5**	15.7	**8.87**	13.3	**7.21**	10.8	**15.7**	23.5	**13.9**	20.9
	11									**12.9**	19.4	**11.5**	17.2

Properties

	$^{1}/_{2}$	$^{3}/_{8}$	$^{5}/_{16}$	$^{1}/_{4}{}^{c}$	$^{1}/_{2}$	$^{7}/_{16}$
A_g (in.^{2})	3.25	2.48	2.09	1.69	3.25	2.87
r_z (in.)	0.633	0.636	0.638	0.639	0.679	0.681

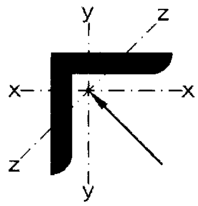
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L3½

Shape		L3½×3½×						L3½×3×					
		³⁄₈		⁵⁄₁₆		¼^c		½		⁷⁄₁₆		³⁄₈	
Wt/ft		8.5		7.2		5.8		10.3		9.1		7.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	53.5	80.4	45.1	67.7	35.2	52.8	64.7	97.2	57.1	85.9	49.6	74.5
	1	52.6	79.1	44.3	66.6	34.6	52.0	63.4	95.3	56.0	84.2	48.6	73.1
	2	50.1	75.3	42.2	63.5	33.1	49.7	59.7	89.8	52.8	79.3	45.8	68.9
	3	46.2	69.4	39.0	58.6	30.6	46.0	54.1	81.3	47.8	71.9	41.6	62.5
	4	41.2	61.9	34.8	52.3	27.5	41.3	47.1	70.8	41.6	62.6	36.2	54.4
	5	35.6	53.5	30.1	45.2	23.9	35.9	39.4	59.2	34.9	52.4	30.4	45.6
	6	29.8	44.8	25.2	37.9	20.2	30.3	31.7	47.6	28.1	42.2	24.5	36.8
	7	24.1	36.2	20.4	30.7	16.5	24.8	24.4	36.7	21.7	32.6	18.9	28.5
	8	18.9	28.3	16.0	24.1	13.1	19.6	18.7	28.1	16.6	24.9	14.5	21.8
	9	14.9	22.4	12.7	19.0	10.3	15.5	14.8	22.2	13.1	19.7	11.5	17.2
	10	12.1	18.1	10.2	15.4	8.36	12.6	12.0	18.0	10.6	16.0	9.28	13.9
	11	10.0	15.0	8.47	12.7	6.91	10.4						

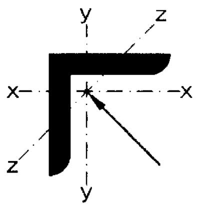
Properties

	L3½×3½×³⁄₈	L3½×3½×⁵⁄₁₆	L3½×3½×¼	L3½×3×½	L3½×3×⁷⁄₁₆	L3½×3×³⁄₈
A_g (in.²)	2.48	2.09	1.69	3.00	2.65	2.30
r_z (in.)	0.683	0.685	0.688	0.618	0.620	0.622

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



L3$^1/_2$

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L3$^1/_2$×3×				L3$^1/_2$×2$^1/_2$×							
		$^5/_{16}$		$^1/_4$^{c}		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$^{c}	
Wt/ft		6.7		5.4		9.4		7.2		6.1		4.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	41.6	62.5	32.5	48.8	59.3	89.1	45.5	68.4	38.4	57.7	30.0	45.0
	1	40.8	61.3	31.9	47.9	57.7	86.7	44.3	66.6	37.4	56.2	29.2	43.9
	2	38.5	57.9	30.1	45.3	53.3	80.0	40.9	61.5	34.6	51.9	27.1	40.7
	3	34.9	52.5	27.5	41.3	46.6	70.0	35.8	53.9	30.3	45.6	23.9	36.0
	4	30.5	45.8	24.1	36.2	38.6	58.0	29.8	44.8	25.2	37.9	20.1	30.2
	5	25.6	38.5	20.4	30.7	30.3	45.6	23.5	35.3	19.9	29.9	16.0	24.1
	6	20.7	31.0	16.6	25.0	22.5	33.9	17.5	26.3	14.9	22.4	12.2	18.3
	7	16.0	24.1	13.1	19.6	16.6	24.9	12.9	19.3	11.0	16.5	8.98	13.5
	8	12.3	18.4	10.0	15.1	12.7	19.1	9.85	14.8	8.40	12.6	6.88	10.3
	9	9.69	14.6	7.92	11.9							5.43	8.17
	10	7.85	11.8	6.41	9.64								

Properties

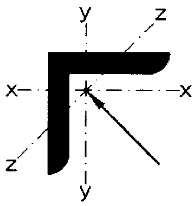
A_g (in.^{2})	1.93	1.56	2.75	2.11	1.78	1.44
r_z (in.)	0.624	0.628	0.532	0.535	0.538	0.541

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L3

Shape		L3×3×											
		1/2		7/16		3/8		5/16		1/4		3/16^c	
Wt/ft		9.35		8.28		7.17		6.04		4.89		3.70	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	**0**	**59.2**	89.0	**52.4**	78.8	**45.4**	68.3	**38.3**	57.5	**31.0**	46.5	**21.4**	32.1
	1	**57.9**	87.1	**51.3**	77.0	**44.4**	66.8	**37.4**	56.3	**30.3**	45.5	**21.0**	31.5
	2	**54.1**	81.4	**47.9**	72.0	**41.5**	62.4	**35.0**	52.6	**28.3**	42.6	**19.7**	29.7
	3	**48.4**	72.7	**42.8**	64.3	**37.1**	55.8	**31.3**	47.1	**25.4**	38.1	**17.8**	26.8
	4	**41.3**	62.1	**36.6**	55.0	**31.7**	47.7	**26.8**	40.3	**21.7**	32.6	**15.5**	23.3
	5	**33.7**	50.7	**29.9**	44.9	**25.9**	39.0	**21.9**	32.9	**17.8**	26.7	**12.9**	19.4
	6	**26.3**	39.5	**23.3**	35.0	**20.3**	30.5	**17.1**	25.8	**13.9**	20.9	**10.4**	15.6
	7	**19.7**	29.6	**17.4**	26.2	**15.2**	22.8	**12.9**	19.3	**10.5**	15.7	**7.97**	12.0
	8	**15.1**	22.7	**13.4**	20.1	**11.6**	17.5	**9.84**	14.8	**8.01**	12.0	**6.10**	9.17
	9	**11.9**	17.9	**10.6**	15.9	**9.18**	13.8	**7.77**	11.7	**6.33**	9.51	**4.82**	7.24

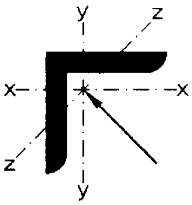
Properties

	1/2	7/16	3/8	5/16	1/4	3/16^c
A_g (in.^{2})	2.75	2.43	2.11	1.78	1.44	1.09
r_z (in.)	0.580	0.580	0.581	0.583	0.585	0.586

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



L3

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L3×2½×											
		½		7/16		3/8		5/16		¼		3/16^c	
Wt/ft		8.53		7.56		6.56		5.54		4.49		3.41	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	53.9	81.0	47.6	71.6	41.4	62.2	36.0	54.1	28.2	42.4	19.6	29.4
	1	52.4	78.7	46.3	69.6	40.2	60.5	35.0	52.6	27.5	41.3	19.1	28.7
	2	48.1	72.3	42.5	63.9	36.9	55.5	32.2	48.3	25.2	37.9	17.7	26.6
	3	41.7	62.7	36.9	55.4	32.1	48.2	27.9	42.0	21.9	33.0	15.6	23.4
	4	34.2	51.3	30.2	45.4	26.3	39.5	22.9	34.4	18.0	27.1	13.0	19.6
	5	26.4	39.7	23.4	35.1	20.3	30.6	17.8	26.7	14.0	21.0	10.4	15.6
	6	19.3	29.0	17.0	25.6	14.9	22.3	13.0	19.5	10.3	15.4	7.84	11.8
	7	14.2	21.3	12.5	18.8	10.9	16.4	9.54	14.3	7.54	11.3	5.77	8.67
	8	10.8	16.3	9.59	14.4	8.36	12.6	7.31	11.0	5.77	8.67	4.42	6.64

Properties

	½	7/16	3/8	5/16	¼	3/16^c
A_g (in.^{2})	2.50	2.21	1.92	1.67	1.31	1.00
r_z (in.)	0.516	0.516	0.517	0.518	0.520	0.521

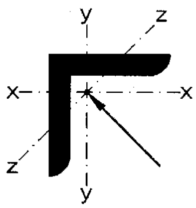
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L3–L2$^1/_2$

Shape		L3×2×										L2$^1/_2$×2$^1/_2$×	
		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$^{c}		$^1/_2$	
Wt/ft		7.70		5.95		5.03		4.09		3.12		7.65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	48.5	72.9	37.3	56.1	31.5	47.3	25.7	38.6	17.7	26.6	48.5	72.8
	1	46.5	69.9	35.8	53.8	30.2	45.4	24.6	37.0	17.1	25.7	46.9	70.5
	2	41.0	61.6	31.6	47.4	26.7	40.1	21.8	32.8	15.3	23.0	42.5	63.9
	3	33.3	50.0	25.6	38.5	21.7	32.6	17.8	26.7	12.8	19.2	36.1	54.3
	4	24.8	37.3	19.1	28.8	16.3	24.4	13.4	20.1	9.87	14.8	28.7	43.2
	5	17.0	25.5	13.1	19.7	11.2	16.8	9.24	13.9	7.10	10.7	21.4	32.2
	6	11.8	17.7	9.12	13.7	7.77	11.7	6.42	9.64	4.94	7.43	15.1	22.7
	7	8.66	13.0	6.70	10.1	5.71	8.58	4.71	7.08	3.63	5.46	11.1	16.7
	8											8.50	12.8

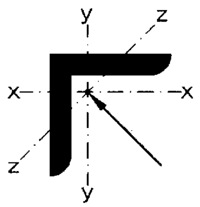
Properties

A_g (in.^{2})	2.25	1.73	1.46	1.19	0.902	2.25
r_z (in.)	0.425	0.426	0.428	0.431	0.435	0.481

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



L2$^1/_2$

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L2$^1/_2$×2$^1/_2$×								L2$^1/_2$×2×	
		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$^{c}		$^3/_8$	
Wt/ft		5.90		4.98		4.04		3.06		5.30	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	37.4	56.1	31.5	47.4	25.6	38.4	19.1	28.7	33.4	50.2
	1	36.1	54.3	30.5	45.9	24.7	37.2	18.5	27.8	32.0	48.1
	2	32.8	49.2	27.7	41.6	22.4	33.7	16.8	25.2	28.1	42.2
	3	27.8	41.8	23.5	35.3	19.1	28.6	14.3	21.5	22.6	34.0
	4	22.1	33.2	18.7	28.1	15.2	22.8	11.4	17.2	16.7	25.2
	5	16.4	24.7	13.9	20.9	11.3	17.0	8.56	12.9	11.4	17.1
	6	11.6	17.4	9.82	14.8	7.98	12.0	6.87	9.13	7.88	11.8
	7	8.53	12.8	7.21	10.8	5.87	8.82	4.46	6.70		
	8	6.53	9.81	5.52	8.30	4.49	6.75	3.42	5.13		

Properties

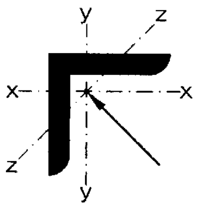
	L2$^1/_2$×2$^1/_2$×$^3/_8$	L2$^1/_2$×2$^1/_2$×$^5/_{16}$	L2$^1/_2$×2$^1/_2$×$^1/_4$	L2$^1/_2$×2$^1/_2$×$^3/_{16}$	L2$^1/_2$×2×$^3/_8$
A_g (in.^{2})	1.73	1.46	1.19	0.900	1.55
r_z (in.)	0.481	0.481	0.482	0.482	0.419

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles



L2½

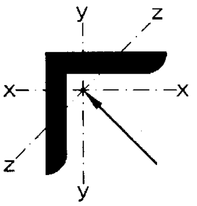
Shape		L2½×2×						L2½×1½×			
		5/16		1/4		3/16^c		1/4		3/16^c	
Wt/ft		4.49		3.65		2.78		3.22		2.47	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	28.2	42.4	22.9	34.3	17.1	25.8	20.2	30.4	15.2	22.8
	1	27.1	40.7	21.9	32.9	16.5	24.7	18.8	28.2	14.1	21.2
	2	23.8	35.7	19.3	29.0	14.5	21.9	15.1	22.6	11.4	17.1
	3	19.2	28.8	15.6	23.4	11.8	17.8	10.4	15.7	8.00	12.0
	4	14.2	21.4	11.6	17.4	8.89	13.4	6.30	9.47	4.89	7.36
	5	9.66	14.5	7.90	11.9	6.13	9.21	4.03	6.06	3.13	4.71
	6	6.71	10.1	5.49	8.25	4.26	6.40				
	7	4.93	7.41	4.03	6.06	3.13	4.70				

Properties

A_g (in.^{2})	1.31	1.06	0.809	0.938	0.715
r_z (in.)	0.420	0.423	0.426	0.321	0.324

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



L2

Table 4–11 (continued)
Available Strength in Axial Compression, kips
Concentrically Loaded Single Angles

F_y = 36 ksi

Shape		L2×2×									
		3/8		5/16		1/4		3/16		1/8^c	
Wt/ft		4.65		3.94		3.21		2.46		1.67	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	29.3	44.1	24.8	37.3	20.2	30.4	15.4	23.2	9.52	14.3
	1	27.9	41.9	23.6	35.4	19.2	28.9	14.7	22.0	9.09	13.7
	2	23.9	35.9	20.2	30.4	16.5	24.8	12.6	19.0	7.94	11.9
	3	18.5	27.9	15.7	23.6	12.8	19.3	9.82	14.8	6.33	9.52
	4	13.0	19.5	11.0	16.5	8.99	13.5	6.91	10.4	4.62	6.94
	5	8.45	12.7	7.15	10.7	5.86	8.81	4.51	6.78	3.09	4.64
	6	5.87	8.82	4.96	7.46	4.07	6.12	3.13	4.71	2.15	3.23

Properties

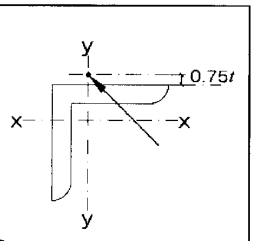
	3/8	5/16	1/4	3/16	1/8^c
A_g (in.^{2})	1.36	1.15	0.938	0.715	0.484
r_z (in.)	0.386	0.386	0.387	0.389	0.391

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–12
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L8

Shape		L8×8×											
		1$^1/_8$		1		$^7/_8$		$^3/_4$		$^5/_8{}^c$		$^9/_{16}{}^c$	
Wt/ft		57.2		51.3		45.3		39.2		33.0		29.8	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	125	187	116	175	107	161	96.6	145	85.1	128	77.5	117
	1	124	187	116	174	107	160	96.5	145	84.9	128	77.4	116
	2	124	186	115	173	106	159	95.9	144	84.4	127	76.9	116
	3	122	184	114	172	105	158	95.0	143	83.6	126	76.2	115
	4	121	182	113	169	104	156	93.7	141	82.4	124	75.2	113
	5	119	178	111	166	102	153	92.0	138	81.0	122	73.9	111
	6	116	175	108	163	99.7	150	90.1	135	79.0	119	71.6	108
	7	113	171	106	159	97.2	146	87.8	132	76.3	115	69.2	104
	8	110	166	103	154	94.5	142	84.6	127	73.5	111	66.7	100
	9	107	161	99.5	150	91.0	137	81.4	122	70.6	106	64.0	96.2
	10	103	155	95.8	144	87.3	131	78.0	117	67.6	102	61.3	92.1
	11	99.1	149	91.6	138	83.5	125	74.5	112	64.5	97.0	58.5	88.0
	12	94.6	142	87.4	131	79.5	120	70.9	107	61.4	92.3	55.7	83.8
	13	90.0	135	83.1	125	75.6	114	67.3	101	58.2	87.5	52.9	79.5
	14	85.4	128	78.8	118	71.6	108	63.7	95.8	55.1	82.8	50.1	75.3
	15	80.8	121	74.5	112	67.6	102	60.1	90.4	51.9	78.1	47.2	71.0
	16	76.2	114	70.1	105	63.6	95.6	56.6	85.0	48.8	73.4	44.4	66.8
	17	71.6	108	65.8	98.9	59.7	89.7	53.0	79.7	45.7	68.7	41.7	62.6
	18	66.9	101	61.5	92.5	55.7	83.7	49.4	74.3	42.6	64.0	38.9	58.5
	19	62.6	94.1	57.5	86.4	52.0	78.2	46.1	69.3	39.7	59.6	36.2	54.5
	20	58.7	88.2	53.8	80.9	48.6	73.1	43.1	64.7	37.0	55.7	33.8	50.8
	21	55.0	82.7	50.4	75.8	45.6	68.5	40.3	60.6	34.6	52.0	31.6	47.5
	22	51.7	77.7	47.4	71.2	42.7	64.2	37.8	56.8	32.4	48.7	29.5	44.4
	23	48.7	73.1	44.5	66.9	40.2	60.4	35.5	53.3	30.4	45.7	27.7	41.6
	24	45.9	68.9	41.9	63.0	37.8	56.8	33.3	50.1	28.6	42.9	26.0	39.1
	25	43.3	65.1	39.6	59.4	35.6	53.5	31.4	47.2	26.9	40.4	24.5	36.8
	26					33.6	50.5	29.6	44.5	25.3	38.1	23.0	34.6

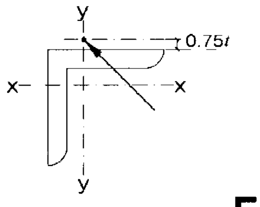
Properties

	1$^1/_8$	1	$^7/_8$	$^3/_4$	$^5/_8{}^c$	$^9/_{16}{}^c$
A_g (in.^{2})	16.8	15.1	13.3	11.5	9.69	8.77
r_z (in.)	1.56	1.56	1.57	1.57	1.58	1.58

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.



L8

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L8×8×		L8×6×									
		$1/2^{c,f}$		1		7/8		3/4		$5/8^{c}$		$9/16^{c}$	
Wt/ft		26.7		44.4		39.3		34.0		28.6		25.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	68.4	103	93.8	141	87.6	132	80.0	120	70.7	106	64.4	96.8
	1	68.2	103	93.5	141	87.3	131	79.6	120	70.4	106	64.1	96.3
	2	67.9	102	92.6	139	86.2	130	78.5	118	69.3	104	63.2	95.0
	3	67.2	101	90.9	137	84.4	127	76.8	115	67.8	102	61.8	92.9
	4	66.4	99.8	88.5	133	82.1	123	74.6	112	65.8	99.0	60.0	90.2
	5	65.3	98.1	85.7	129	79.3	119	72.1	108	63.5	95.5	57.9	87.0
	6	63.5	95.5	82.4	124	76.3	115	69.2	104	60.9	91.6	55.6	83.5
	7	61.4	92.2	78.9	119	72.9	110	66.1	99.3	58.1	87.3	53.0	79.7
	8	59.1	88.8	75.2	113	69.4	104	62.8	94.3	55.1	82.9	50.3	75.6
	9	56.8	85.3	71.3	107	65.7	98.7	59.4	89.2	52.1	78.2	47.5	71.4
	10	54.4	81.8	67.2	101	61.9	93.0	55.9	84.0	48.9	73.5	44.7	67.2
	11	52.0	78.1	63.2	95.0	58.1	87.3	52.3	78.7	45.8	68.8	41.8	62.9
	12	49.5	74.4	59.1	88.9	54.3	81.5	48.8	73.4	42.7	64.1	39.0	58.6
	13	47.0	70.7	55.1	82.8	50.5	75.9	45.4	68.2	39.6	59.5	36.2	54.4
	14	44.6	67.0	51.1	76.8	46.8	70.3	42.0	63.1	36.6	55.0	33.5	50.3
	15	42.1	63.3	47.2	70.9	43.1	64.8	38.6	58.0	33.6	50.5	30.8	46.3
	16	39.7	59.6	43.6	65.5	39.8	59.8	35.6	53.5	30.9	46.4	28.3	42.5
	17	37.3	56.0	40.4	60.7	36.8	55.3	32.9	49.4	28.5	42.8	26.1	39.2
	18	34.9	52.5	37.5	56.4	34.1	51.3	30.4	45.7	26.3	39.6	24.1	36.2
	19	32.5	48.9	34.9	52.4	31.7	47.6	28.2	42.4	24.4	36.6	22.3	33.5
	20	30.3	45.6	32.5	48.9	29.5	44.4	26.2	39.4	22.6	34.0	20.7	31.1
	21	28.3	42.6	30.4	45.7	27.5	41.4	24.5	36.8	21.1	31.7	19.2	28.9
	22	26.5	39.8										
	23	24.8	37.3										
	24	23.3	35.0										
	25	21.9	32.9										
	26	20.6	31.0										

Properties

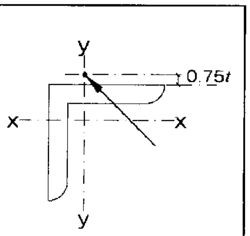
	L8×8×1/2	L8×6×1	7/8	3/4	5/8	9/16
A_g (in.^{2})	7.84	13.1	11.5	9.99	8.41	7.61
r_z (in.)	1.59	1.28	1.28	1.29	1.29	1.30

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L8

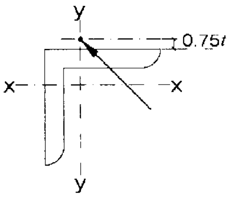
Shape		L8×6×				L8×4×							
		$^1/_2{}^{c,f}$		$^7/_{16}{}^{c,f}$		1		$^7/_8$		$^3/_4$		$^5/_8{}^c$	
Wt/ft		23.2		20.4		37.6		33.3		28.9		24.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	57.4	86.3	49.6	74.6	58.5	87.9	55.3	83.1	51.5	77.5	46.9	70.4
	1	57.1	85.9	49.4	74.2	58.0	87.2	54.9	82.5	51.1	76.9	46.5	69.8
	2	56.4	84.7	48.7	73.2	56.8	85.4	53.7	80.6	49.9	75.1	45.3	68.2
	3	55.1	82.9	47.7	71.7	54.9	82.5	51.8	77.8	48.1	72.3	43.6	65.5
	4	53.6	80.5	46.4	69.7	52.4	78.8	49.3	74.1	45.7	68.7	41.3	62.1
	5	51.7	77.7	44.8	67.3	49.5	74.5	46.5	69.9	43.0	64.6	38.7	58.2
	6	49.6	74.6	43.0	64.6	46.3	69.6	43.3	65.1	40.0	60.1	35.9	54.0
	7	47.4	71.2	41.1	61.7	42.9	64.5	40.0	60.2	36.8	55.3	32.9	49.5
	8	45.0	67.6	39.0	58.7	39.4	59.2	36.6	55.1	33.6	50.5	30.0	45.0
	9	42.5	63.9	36.9	55.5	35.9	53.9	33.3	50.0	30.4	45.6	27.0	40.6
	10	40.0	60.1	34.8	52.3	32.3	48.6	29.9	44.9	27.2	40.9	24.1	36.3
	11	37.5	56.3	32.6	49.1	29.1	43.8	26.9	40.4	24.4	36.6	21.6	32.4
	12	35.0	52.6	30.5	45.9	26.4	39.6	24.2	36.4	21.9	33.0	19.3	29.1
	13	32.5	48.9	28.4	42.7	23.9	36.0	22.0	33.0	19.8	29.8	17.4	26.2
	14	30.1	45.3	26.4	39.7	21.8	32.8	20.0	30.0	18.0	27.0	15.8	23.7
	15	27.8	41.8	24.4	36.7								
	16	25.5	38.4	22.5	33.9								
	17	23.5	35.3	20.7	31.2								
	18	21.7	32.6	19.1	28.8								
	19	20.1	30.2	17.7	26.6								
	20	18.6	28.0	16.4	24.7								
	21	17.3	26.0	15.3	23.0								

Properties

A_g (in.^{2})	6.80	5.99	11.1	9.79	8.49	7.16
r_z (in.)	1.30	1.31	0.844	0.846	0.850	0.856

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



L8–L7

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L8×4×						L7×4×					
		$^9/_{16}$^{c}		$^1/_2$^{c,f}		$^7/_{16}$^{c,f}		$^3/_4$		$^5/_8$		$^1/_2$^{c}	
Wt/ft		22.1		19.7		17.4		26.2		22.1		17.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	**0**	43.5	65.4	39.5	59.4	34.7	52.2	48.7	73.2	44.3	66.6	38.2	57.4
	1	43.1	64.8	39.2	58.9	34.4	51.8	48.3	72.6	43.9	66.0	37.8	56.9
	2	42.1	63.3	38.2	57.5	33.6	50.5	47.1	70.8	42.8	64.3	36.9	55.4
	3	40.4	60.8	36.7	55.2	32.3	48.6	45.4	68.2	41.1	61.8	35.4	53.2
	4	38.3	57.6	34.8	52.3	30.7	46.1	43.1	64.8	39.0	58.6	33.5	50.3
	5	35.9	54.0	32.6	49.0	28.8	43.3	40.5	60.9	36.5	54.9	31.3	47.0
	6	33.3	50.0	30.2	45.5	26.8	40.2	37.6	56.6	33.8	50.8	28.9	43.5
	7	30.5	45.9	27.8	41.8	24.7	37.1	34.6	52.1	31.0	46.6	26.5	39.8
	8	27.8	41.7	25.3	38.1	22.6	33.9	31.6	47.5	28.2	42.4	24.0	36.1
	9	25.1	37.7	22.9	34.4	20.5	30.8	28.5	42.9	25.4	38.2	21.6	32.5
	10	22.4	33.7	20.5	30.9	18.4	27.7	25.6	38.4	22.7	34.1	19.3	29.0
	11	20.0	30.1	18.3	27.5	16.5	24.7	22.9	34.4	20.2	30.4	17.1	25.7
	12	17.9	26.9	16.4	24.6	14.7	22.1	20.6	30.9	18.1	27.2	15.3	23.0
	13	16.1	24.3	14.7	22.2	13.2	19.9	18.5	27.9	16.3	24.5	13.7	20.7
	14	14.6	21.9	13.3	20.0	12.0	18.0	16.8	25.3	14.7	22.1	12.4	18.6

Properties

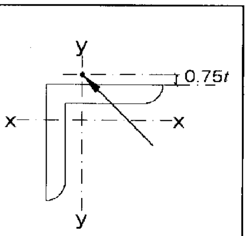
A_g (in.^{2})	6.49	5.80	5.11	7.70	6.50	5.26
r_z (in.)	0.859	0.863	0.867	0.855	0.860	0.866

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

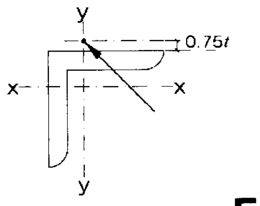


L7–L6

Shape		L7×4×				L6×6×							
		7/16^{c,f}		3/8^{c,f}		1		7/8		3/4		5/8	
Wt/ft		15.8		13.6		37.5		33.2		28.8		24.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length KL (ft) with respect to least radius of gyration r_z	**0**	34.0	51.1	28.9	43.4	76.7	115	71.3	107	65.1	97.9	58.1	87.3
	1	33.7	50.6	28.6	43.1	76.5	115	71.1	107	64.9	97.6	57.9	87.0
	2	32.8	49.3	27.9	42.0	75.7	114	70.4	106	64.2	96.6	57.3	86.1
	3	31.5	47.3	26.8	40.3	74.5	112	69.2	104	63.2	94.9	56.3	84.6
	4	29.8	44.8	25.4	38.2	72.8	109	67.6	102	61.7	92.7	54.9	82.6
	5	27.8	41.8	23.8	35.8	70.7	106	65.5	98.5	59.8	89.9	53.2	80.0
	6	25.7	38.7	22.1	33.2	68.2	102	63.2	94.9	57.6	86.5	51.2	76.9
	7	23.6	35.5	20.3	30.5	65.3	98.2	60.5	90.9	55.1	82.8	48.6	73.0
	8	21.4	32.2	18.5	27.8	62.2	93.5	57.6	86.5	52.1	78.3	45.8	68.9
	9	19.3	29.1	16.8	25.2	58.9	88.5	54.3	81.6	49.0	73.6	43.0	64.6
	10	17.3	26.0	15.1	22.7	55.4	83.3	50.8	76.4	45.7	68.8	40.1	60.3
	11	15.4	23.1	13.5	20.2	51.6	77.6	47.3	71.1	42.5	63.9	37.2	55.9
	12	13.7	20.6	12.0	18.1	47.8	71.9	43.7	65.7	39.3	59.0	34.3	51.6
	13	12.3	18.5	10.8	16.2	44.1	66.2	40.2	60.4	36.1	54.2	31.5	47.3
	14	11.1	16.7	9.71	14.6	40.4	60.7	36.8	55.3	32.9	49.5	28.7	43.1
	15					37.1	55.7	33.7	50.7	30.1	45.3	26.2	39.4
	16					34.1	51.3	31.0	46.6	27.6	41.5	24.0	36.1
	17					31.5	47.3	28.6	42.9	25.4	38.2	22.0	33.1
	18					29.1	43.8	26.4	39.7	23.5	35.3	20.3	30.5
	19					27.0	40.6	24.4	36.7	21.7	32.6	18.8	28.2
Properties													
A_g (in.^{2})		4.63		4.00		11.0		9.75		8.46		7.13	
r_z (in.)		0.869		0.873		1.17		1.17		1.17		1.17	

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.



L6

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L6×6×										L6×4×	
		9/16		1/2		$7/16^c$		$3/8^{c,f}$		$5/16^{c,f}$		7/8	
Wt/ft		22.0		19.6		17.3		14.9		12.5		27.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	54.2	81.4	49.9	75.0	44.8	67.4	38.3	57.5	29.4	44.1	49.3	74.2
	1	54.0	81.1	49.8	74.8	44.7	67.2	38.1	57.3	29.3	44.0	49.0	73.7
	2	53.4	80.3	49.2	74.0	44.2	66.5	37.8	56.7	29.0	43.6	47.9	72.0
	3	52.5	78.9	48.4	72.7	43.5	65.3	37.1	55.8	28.6	43.0	46.2	69.4
	4	51.2	77.0	47.2	70.9	42.4	63.7	36.3	54.5	28.0	42.1	43.9	66.0
	5	49.6	74.6	45.5	68.4	40.6	61.0	34.7	52.2	27.3	41.0	41.3	62.1
	6	47.4	71.2	43.3	65.1	38.6	58.0	33.1	49.7	26.4	39.7	38.4	57.8
	7	44.9	67.5	41.0	61.7	36.6	54.9	31.3	47.1	25.3	38.1	35.4	53.2
	8	42.4	63.7	38.7	58.1	34.4	51.7	29.5	44.4	23.9	35.9	32.4	48.6
	9	39.7	59.7	36.2	54.4	32.2	48.5	27.7	41.6	22.5	33.8	29.3	44.0
	10	37.0	55.6	33.7	50.7	30.0	45.2	25.8	38.8	21.0	31.6	26.2	39.4
	11	34.3	51.6	31.2	47.0	27.8	41.8	24.0	36.1	19.6	29.4	23.5	35.4
	12	31.6	47.5	28.8	43.3	25.7	38.6	22.2	33.3	18.2	27.3	21.2	31.8
	13	29.0	43.6	26.3	39.6	23.5	35.3	20.4	30.6	16.8	25.2	19.1	28.7
	14	26.4	39.6	24.0	36.0	21.4	32.1	18.6	27.9	15.4	23.1	17.3	26.1
	15	24.1	36.2	21.8	32.8	19.5	29.3	16.9	25.4	14.0	21.1		
	16	22.0	33.1	20.0	30.0	17.8	26.7	15.4	23.2	12.8	19.3		
	17	20.2	30.4	18.3	27.5	16.3	24.5	14.1	21.2	11.7	17.6		
	18	18.6	28.0	16.9	25.3	15.0	22.5	13.0	19.5	10.8	16.2		
	19	17.2	25.9	15.6	23.4	13.8	20.8	12.0	18.0	9.91	14.9		

Properties

	9/16	1/2	$7/16^c$	$3/8^{c,f}$	$5/16^{c,f}$	7/8
A_g (in.^{2})	6.45	5.77	5.08	4.38	3.67	7.98
r_z (in.)	1.18	1.18	1.18	1.19	1.19	0.854

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

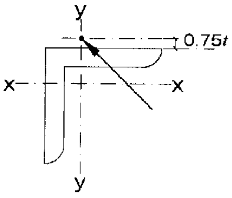
[c] Shape is slender for compression with $F_y = 36$ ksi.

[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L6

Shape		L6×4×									
		$^3/_4$		$^5/_8$		$^9/_{16}$		$^1/_2$		$^7/_{16}$[c]	
Wt/ft		23.6		19.9		18.1		16.2		14.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	46.1	69.3	41.9	63.0	39.4	59.3	36.6	55.1	33.1	49.8
	1	45.7	68.7	41.5	62.4	39.0	58.7	36.3	54.5	32.8	49.3
	2	44.5	66.9	40.4	60.7	38.0	57.1	35.3	53.0	31.9	47.9
	3	42.8	64.3	38.8	58.3	36.4	54.7	33.8	50.8	30.5	45.8
	4	40.6	61.0	36.7	55.2	34.4	51.7	31.9	47.9	28.8	43.3
	5	38.1	57.2	34.3	51.6	32.2	48.3	29.8	44.7	26.8	40.3
	6	35.3	53.1	31.8	47.7	29.7	44.6	27.4	41.2	24.7	37.2
	7	32.5	48.8	29.1	43.7	27.2	40.8	25.0	37.6	22.6	33.9
	8	29.6	44.4	26.4	39.7	24.6	37.0	22.7	34.0	20.4	30.7
	9	26.7	40.1	23.7	35.7	22.1	33.2	20.3	30.5	18.3	27.5
	10	23.8	35.8	21.1	31.8	19.6	29.5	18.0	27.1	16.2	24.4
	11	21.3	32.0	18.8	28.3	17.4	26.2	16.0	24.0	14.3	21.6
	12	19.1	28.7	16.8	25.3	15.6	23.4	14.2	21.4	12.8	19.2
	13	17.2	25.9	15.1	22.7	14.0	21.0	12.7	19.1	11.4	17.2
	14	15.6	23.4	13.6	20.5	12.6	18.9	11.5	17.2	10.3	15.4

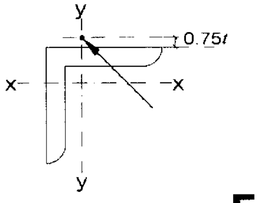
Properties

	$^3/_4$	$^5/_8$	$^9/_{16}$	$^1/_2$	$^7/_{16}$
A_g (in.^{2})	6.94	5.86	5.31	4.75	4.18
r_z (in.)	0.856	0.859	0.861	0.864	0.867

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



L6

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L6×4×				L6×3$^{1}/_{2}$×					
		3/8^{c,f}		5/16^{c,f}		1/2		3/8^{c,f}		5/16^{c,f}	
Wt/ft		12.3		10.3		15.4		11.7		9.83	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	28.7	43.1	23.4	35.2	32.1	48.2	25.5	38.4	21.0	31.5
	1	28.4	42.7	23.2	34.9	31.7	47.6	25.2	37.9	20.7	31.2
	2	27.6	41.5	22.6	33.9	30.7	46.1	24.4	36.7	20.1	30.2
	3	26.5	39.8	21.6	32.5	29.1	43.7	23.1	34.8	19.0	28.6
	4	25.0	37.5	20.4	30.7	27.1	40.8	21.6	32.4	17.8	26.8
	5	23.3	35.0	19.1	28.7	24.9	37.5	19.8	29.8	16.4	24.7
	6	21.5	32.3	17.6	26.5	22.6	34.0	17.9	27.0	15.0	22.5
	7	19.6	29.5	16.2	24.3	20.2	30.4	16.1	24.2	13.5	20.3
	8	17.8	26.7	14.7	22.1	17.9	26.9	14.3	21.5	12.1	18.1
	9	15.9	24.0	13.3	20.0	15.7	23.6	12.6	18.9	10.7	16.1
	10	14.2	21.4	11.9	17.9	13.8	20.7	11.0	16.5	9.37	14.1
	11	12.6	18.9	10.6	16.0	12.2	18.3	9.68	14.5	8.25	12.4
	12	11.2	16.8	9.43	14.2	10.8	16.2	8.57	12.9	7.31	11.0
	13	10.0	15.0	8.43	12.7						
	14	8.98	13.5	7.58	11.4						

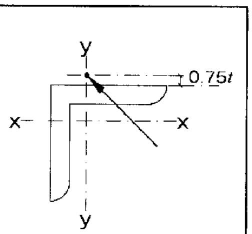
Properties					
A_g (in.^{2})	3.61	3.03	4.52	3.44	2.89
r_z (in.)	0.870	0.874	0.756	0.763	0.767

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



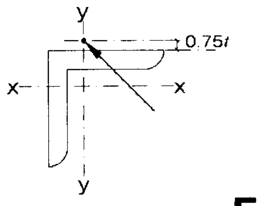
L5

Shape		L5×5×											
		7/8		3/4		5/8		1/2		7/16		$3/8^c$	
Wt/ft		27.3		23.7		20.1		16.3		14.4		12.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	54.8	82.4	50.5	75.9	45.4	68.3	39.5	59.3	36.1	54.3	32.2	48.4
	1	54.6	82.0	50.2	75.5	45.2	67.9	39.3	59.0	35.9	54.0	32.0	48.1
	2	53.8	80.8	49.5	74.4	44.5	66.9	38.7	58.2	35.4	53.2	31.5	47.4
	3	52.5	79.0	48.3	72.6	43.5	65.3	37.7	56.7	34.5	51.9	30.8	46.2
	4	50.8	76.4	46.7	70.2	42.0	63.1	36.4	54.8	33.3	50.1	29.4	44.2
	5	48.7	73.3	44.7	67.2	40.2	60.4	34.7	52.2	31.5	47.3	27.7	41.7
	6	46.3	69.6	42.5	63.8	38.1	57.2	32.6	48.9	29.5	44.3	26.0	39.0
	7	43.6	65.6	39.9	60.0	35.5	53.3	30.3	45.5	27.4	41.1	24.1	36.2
	8	40.7	61.2	37.1	55.7	32.8	49.3	28.0	42.0	25.2	37.9	22.2	33.4
	9	37.6	56.6	34.1	51.2	30.1	45.2	25.6	38.5	23.1	34.7	20.3	30.5
	10	34.4	51.7	31.1	46.7	27.4	41.2	23.2	34.9	20.9	31.5	18.4	27.7
	11	31.1	46.8	28.1	42.2	24.7	37.1	20.9	31.4	18.8	28.3	16.6	24.9
	12	28.1	42.2	25.2	37.9	22.1	33.3	18.7	28.1	16.8	25.3	14.8	22.2
	13	25.4	38.2	22.8	34.3	19.9	30.0	16.8	25.3	15.1	22.7	13.2	19.9
	14	23.1	34.7	20.6	31.0	18.0	27.1	15.2	22.8	13.6	20.4	11.9	17.9
	15	21.0	31.6	18.8	28.2	16.4	24.6	13.7	20.6	12.3	18.5	10.8	16.2
	16	19.2	28.9	17.1	25.8	14.9	22.4	12.5	18.8	11.2	16.8	9.77	14.7

Properties						
A_g (in.^{2})	8.02	6.98	5.90	4.79	4.22	3.65
r_z (in.)	0.971	0.972	0.975	0.980	0.983	0.986

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.



L5

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L5×5×		L5×3½×									
		$^5/_{16}{}^{c,f}$		$^3/_4$		$^5/_8$		$^1/_2$		$^3/_8{}^{c}$		$^5/_{16}{}^{c,f}$	
Wt/ft		10.4		19.8		16.8		13.6		10.4		8.72	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	26.8	40.3	36.7	55.2	33.8	50.8	30.0	45.1	24.7	37.1	20.9	31.3
	1	26.7	40.1	36.4	54.7	33.4	50.3	29.6	44.5	24.4	36.6	20.6	30.9
	2	26.3	39.5	35.4	53.2	32.3	48.6	28.5	42.9	23.5	35.3	19.8	29.8
	3	25.7	38.6	33.7	50.6	30.7	46.1	27.0	40.6	22.2	33.3	18.7	28.2
	4	24.6	36.9	31.5	47.4	28.7	43.1	25.1	37.8	20.6	30.9	17.4	26.1
	5	23.2	34.8	29.1	43.8	26.4	39.6	23.0	34.6	18.8	28.2	15.9	23.9
	6	21.7	32.6	26.6	39.9	24.0	36.0	20.8	31.3	16.9	25.4	14.3	21.6
	7	20.2	30.3	23.9	36.0	21.5	32.3	18.6	28.0	15.1	22.6	12.8	19.2
	8	18.6	28.0	21.3	32.0	19.0	28.6	16.4	24.7	13.2	19.9	11.3	16.9
	9	17.1	25.7	18.7	28.2	16.7	25.1	14.3	21.5	11.5	17.3	9.83	14.8
	10	15.5	23.4	16.5	24.9	14.7	22.0	12.5	18.8	10.0	15.0	8.54	12.8
	11	14.0	21.1	14.7	22.1	13.0	19.5	11.0	16.6	8.77	13.2	7.48	11.2
	12	12.6	18.9	13.1	19.7	11.5	17.3	9.77	14.7	7.74	11.6	6.59	9.91
	13	11.2	16.9										
	14	10.1	15.2										
	15	9.11	13.7										
	16	8.26	12.4										

Properties						
A_g (in.²)	3.07	5.82	4.93	4.00	3.05	2.56
r_z (in.)	0.990	0.744	0.746	0.750	0.755	0.758

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

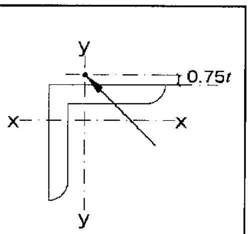
[c] Shape is slender for compression with F_y = 36 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L5

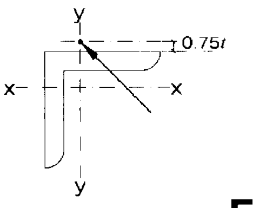
Shape	L5×3½×		L5×3×									
	¼^{c,f}		½		7/16		3/8^{c}		5/16^{c,f}		¼^{c,f}	
Wt/ft	7.03		12.8		11.3		9.74		8.19		6.60	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z												
0	16.1	24.3	25.2	37.9	23.4	35.2	21.1	31.8	18.0	27.1	14.0	21.1
1	15.9	24.0	24.8	37.3	23.0	34.6	20.8	31.3	17.7	26.7	13.8	20.8
2	15.4	23.1	23.8	35.7	22.0	33.1	19.9	29.8	16.9	25.5	13.2	19.8
3	14.6	21.9	22.2	33.4	20.5	30.8	18.5	27.8	15.8	23.7	12.3	18.5
4	13.5	20.3	20.3	30.5	18.7	28.1	16.8	25.3	14.3	21.5	11.3	16.9
5	12.4	18.6	18.2	27.4	16.8	25.2	15.0	22.6	12.8	19.3	10.1	15.2
6	11.2	16.9	16.1	24.2	14.7	22.1	13.2	19.8	11.3	16.9	9.01	13.5
7	10.1	15.2	14.0	21.0	12.8	19.2	11.4	17.1	9.78	14.7	7.89	11.9
8	8.98	13.5	12.0	18.0	10.9	16.4	9.71	14.6	8.36	12.6	6.83	10.3
9	7.91	11.9	10.3	15.5	9.39	14.1	8.33	12.5	7.17	10.8	5.85	8.79
10	6.90	10.4	9.00	13.5	8.15	12.2	7.22	10.9	6.20	9.32	5.06	7.60
11	6.04	9.08										
12	5.33	8.01										

Properties

	L5×3½×¼	L5×3×½	L5×3×7/16	L5×3×3/8	L5×3×5/16	L5×3×¼
A_g (in.^{2})	2.07	3.75	3.31	2.86	2.41	1.94
r_z (in.)	0.761	0.642	0.644	0.646	0.649	0.652

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.



L4

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape	L4×4×											
	3/4		5/8		1/2		7/16		3/8		5/16^c	
Wt/ft	18.5		15.7		12.7		11.2		9.72		8.16	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z												
0	36.1	54.2	32.8	49.2	28.8	43.3	26.5	39.8	23.9	35.9	21.0	31.6
1	35.8	53.8	32.5	48.9	28.6	42.9	26.3	39.5	23.7	35.6	20.8	31.3
2	35.0	52.7	31.8	47.8	27.9	41.9	25.6	38.5	23.2	34.8	20.3	30.6
3	33.8	50.8	30.6	46.0	26.8	40.3	24.7	37.1	22.2	33.4	19.5	29.3
4	32.1	48.3	29.0	43.6	25.4	38.2	23.3	35.1	20.9	31.4	18.1	27.3
5	30.1	45.2	27.2	40.8	23.7	35.6	21.6	32.4	19.2	28.9	16.6	25.0
6	27.8	41.8	25.0	37.6	21.6	32.4	19.6	29.5	17.5	26.3	15.1	22.7
7	25.4	38.1	22.6	34.0	19.4	29.2	17.6	26.5	15.7	23.6	13.5	20.3
8	22.8	34.2	20.2	30.4	17.3	26.0	15.6	23.5	13.9	20.9	11.9	18.0
9	20.1	30.2	17.8	26.7	15.1	22.7	13.7	20.6	12.1	18.2	10.4	15.6
10	17.7	26.6	15.6	23.4	13.2	19.9	11.9	17.9	10.5	15.8	9.03	13.6
11	15.7	23.6	13.8	20.7	11.6	17.5	10.5	15.8	9.24	13.9	7.90	11.9
12	14.0	21.0	12.2	18.4	10.3	15.5	9.26	13.9	8.15	12.3	6.96	10.5
13											6.17	9.27

Properties						
A_g (in.^{2})	5.43	4.61	3.75	3.30	2.86	2.40
r_z (in.)	0.774	0.774	0.776	0.777	0.779	0.781

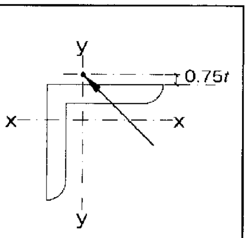
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L4

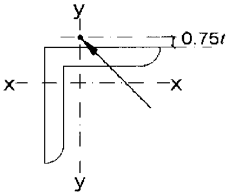
Shape		L4×4×		L4×3½×								L4×3×	
		$^{1}/_{4}$^{c,f}		½		$^{3}/_{8}$		$^{5}/_{16}$^{c}		$^{1}/_{4}$^{c,f}		$^{5}/_{8}$	
Wt/ft		6.58		11.9		9.10		7.65		6.18		13.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	**0**	16.9	25.3	26.8	40.3	22.6	33.9	20.0	30.1	16.3	24.5	25.8	38.8
	1	16.7	25.2	26.5	39.9	22.3	33.6	19.8	29.8	16.0	24.0	25.5	38.3
	2	16.4	24.6	25.7	38.7	21.5	32.4	18.9	28.4	15.2	22.9	24.5	36.9
	3	15.7	23.5	24.4	36.7	20.2	30.3	17.6	26.5	14.2	21.4	22.9	34.5
	4	14.6	21.9	22.6	34.0	18.6	28.0	16.2	24.4	13.1	19.7	21.0	31.6
	5	13.4	20.1	20.6	30.9	16.9	25.4	14.7	22.1	11.9	17.9	18.9	28.4
	6	12.2	18.3	18.5	27.8	15.1	22.7	13.1	19.7	10.6	16.0	16.7	25.0
	7	10.9	16.4	16.4	24.6	13.3	20.0	11.5	17.3	9.37	14.1	14.5	21.7
	8	9.69	14.6	14.3	21.4	11.5	17.3	9.96	15.0	8.15	12.3	12.4	18.7
	9	8.50	12.8	12.3	18.5	9.91	14.9	8.53	12.8	6.99	10.5	10.7	16.1
	10	7.36	11.1	10.7	16.1	8.59	12.9	7.37	11.1	6.02	9.05	9.34	14.0
	11	6.43	9.66	9.41	14.1	7.50	11.3	6.42	9.66	5.23	7.86		
	12	5.65	8.49					5.64	8.47	4.58	6.88		
	13	5.00	7.51										

Properties

	L4×4×¼	L4×3½×½	L4×3½×⅜	L4×3½×$^{5}/_{16}$	L4×3½×¼	L4×3×⅝
A_g (in.^{2})	1.93	3.50	2.68	2.25	1.82	3.99
r_z (in.)	0.783	0.716	0.719	0.721	0.723	0.631

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with $F_y = 36$ ksi.
[f] Shape exceeds compact limit for flexure with $F_y = 36$ ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



L4–L3½

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L4×3×								L3½×3½×			
		½		⅜		$^5/_{16}$^{c}		¼^{c,f}		½		$^7/_{16}$	
Wt/ft		11.1		8.47		7.12		5.75		11.1		9.82	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	23.2	34.9	19.8	29.7	17.5	26.2	14.2	21.3	24.0	36.1	22.2	33.4
	1	22.9	34.4	19.4	29.1	17.1	25.7	13.9	20.9	23.8	35.7	22.0	33.0
	2	21.9	32.9	18.4	27.7	16.2	24.4	13.2	19.8	23.1	34.7	21.3	32.0
	3	20.3	30.6	17.1	25.6	15.0	22.5	12.2	18.3	21.9	33.0	20.3	30.5
	4	18.5	27.9	15.4	23.2	13.5	20.4	11.0	16.6	20.5	30.8	18.9	28.4
	5	16.6	24.9	13.7	20.6	12.0	18.0	9.78	14.7	18.7	28.2	17.1	25.8
	6	14.5	21.9	12.0	18.0	10.4	15.7	8.53	12.8	16.7	25.1	15.3	22.9
	7	12.5	18.9	10.3	15.4	8.91	13.4	7.31	11.0	14.7	22.0	13.4	20.1
	8	10.7	16.1	8.68	13.1	7.51	11.3	6.17	9.28	12.6	19.0	11.5	17.3
	9	9.19	13.8	7.42	11.2	6.40	9.62	5.24	7.88	10.9	16.4	9.87	14.8
	10	7.97	12.0	6.40	9.62	5.50	8.27	4.50	6.76	9.45	14.2	8.55	12.8
	11									8.26	12.4	7.46	11.2

Properties

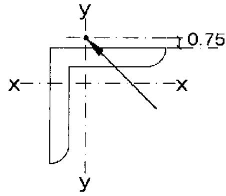
	L4×3×½	L4×3×⅜	L4×3×$^5/_{16}$	L4×3×¼	L3½×3½×½	L3½×3½×$^7/_{16}$
A_g (in.^{2})	3.25	2.49	2.09	1.69	3.27	2.89
r_z (in.)	0.633	0.636	0.638	0.639	0.679	0.681

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L3½

Shape		L3½×3½×						L3½×3×					
		3/8		5/16		1/4^c		1/2		7/16		3/8	
Wt/ft		8.51		7.16		5.79		10.3		9.09		7.88	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	20.2	30.3	17.9	26.9	15.1	22.6	21.8	32.8	20.4	30.6	18.7	28.1
	1	20.0	30.0	17.7	26.6	14.9	22.4	21.6	32.4	20.1	30.2	18.4	27.7
	2	19.4	29.1	17.2	25.8	14.5	21.7	20.7	31.1	19.3	29.0	17.6	26.4
	3	18.4	27.7	16.3	24.4	13.5	20.3	19.3	29.1	17.9	26.9	16.2	24.4
	4	17.0	25.6	14.9	22.4	12.4	18.6	17.6	26.4	16.2	24.3	14.7	22.0
	5	15.4	23.1	13.4	20.2	11.1	16.7	15.6	23.5	14.4	21.6	13.0	19.5
	6	13.7	20.5	11.9	17.9	9.85	14.8	13.6	20.5	12.5	18.8	11.3	16.9
	7	11.9	17.9	10.4	15.6	8.59	12.9	11.7	17.5	10.7	16.0	9.58	14.4
	8	10.2	15.4	8.87	13.3	7.36	11.1	9.91	14.9	9.03	13.6	8.08	12.1
	9	8.77	13.2	7.58	11.4	6.27	9.42	8.49	12.8	7.72	11.6	6.88	10.3
	10	7.58	11.4	6.53	9.82	5.39	8.10	7.34	11.0	6.66	10.0	5.92	8.90
	11	6.60	9.92	5.68	8.54	4.67	7.02						

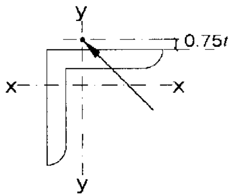
Properties

A_g (in.^{2})	2.50	2.10	1.70	3.02	2.67	2.32
r_z (in.)	0.683	0.685	0.688	0.618	0.620	0.622

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.



L3$^1/_2$

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L3$^1/_2$×3×				L3$^1/_2$×2$^1/_2$×							
		$^5/_{16}$		$^1/_4{}^c$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4{}^c$	
Wt/ft		6.65		5.38		9.41		7.23		6.10		4.94	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	16.7	25.2	14.2	21.4	18.1	27.2	15.7	23.7	14.1	21.2	12.0	18.0
	1	16.5	24.8	13.9	20.8	17.8	26.7	15.4	23.1	13.8	20.7	11.7	17.5
	2	15.6	23.4	13.1	19.6	16.8	25.2	14.4	21.6	12.8	19.3	10.9	16.3
	3	14.3	21.6	12.0	18.0	15.3	22.9	13.0	19.5	11.6	17.4	9.79	14.7
	4	12.9	19.4	10.8	16.2	13.5	20.3	11.4	17.2	10.2	15.3	8.56	12.9
	5	11.4	17.1	9.51	14.3	11.7	17.6	9.81	14.7	8.67	13.0	7.30	11.0
	6	9.86	14.8	8.23	12.4	9.86	14.8	8.20	12.3	7.22	10.9	6.08	9.13
	7	8.37	12.6	6.99	10.5	8.23	12.4	6.78	10.2	5.94	8.93	4.98	7.48
	8	7.03	10.6	5.85	8.79	6.94	10.4	5.68	8.54	4.96	7.45	4.13	6.21
	9	5.97	8.97	4.95	7.44							3.48	5.23
	10	5.12	7.70	4.23	6.36								

Properties

	L3$^1/_2$×3×$^5/_{16}$	L3$^1/_2$×3×$^1/_4$	L3$^1/_2$×2$^1/_2$×$^1/_2$	L3$^1/_2$×2$^1/_2$×$^3/_8$	L3$^1/_2$×2$^1/_2$×$^5/_{16}$	L3$^1/_2$×2$^1/_2$×$^1/_4$
A_g (in.^{2})	1.95	1.58	2.76	2.12	1.79	1.45
r_z (in.)	0.624	0.628	0.532	0.535	0.538	0.541

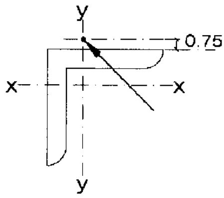
ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L3

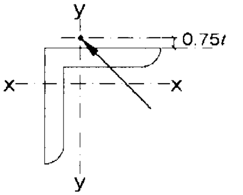
Shape		L3×3×											
		1/2		7/16		3/8		5/16		1/4		3/16^{c,f}	
Wt/ft		9.35		8.28		7.17		6.04		4.89		3.70	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	19.1	28.7	17.7	26.6	16.2	24.3	14.4	21.7	12.4	18.6	9.47	14.2
	1	18.8	28.3	17.5	26.3	16.0	24.0	14.2	21.4	12.2	18.3	9.34	14.0
	2	18.1	27.2	16.8	25.2	15.3	23.0	13.6	20.5	11.7	17.6	8.97	13.5
	3	16.9	25.4	15.7	23.6	14.3	21.5	12.7	19.1	10.7	16.1	8.17	12.3
	4	15.4	23.2	14.3	21.5	12.9	19.4	11.3	17.0	9.55	14.4	7.28	10.9
	5	13.7	20.7	12.6	18.9	11.3	17.0	9.91	14.9	8.32	12.5	6.36	9.56
	6	11.9	17.8	10.8	16.3	9.71	14.6	8.47	12.7	7.08	10.6	5.44	8.17
	7	9.99	15.0	9.09	13.7	8.12	12.2	7.06	10.6	5.88	8.84	4.54	6.83
	8	8.44	12.7	7.66	11.5	6.82	10.3	5.91	8.88	4.90	7.37	3.77	5.67
	9	7.21	10.8	6.52	9.80	5.79	8.70	5.00	7.51	4.14	6.22	3.17	4.76

Properties

	1/2	7/16	3/8	5/16	1/4	3/16
A_g (in.^{2})	2.75	2.43	2.11	1.78	1.44	1.09
r_z (in.)	0.580	0.580	0.581	0.583	0.585	0.586

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.



L3

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L3×2$^1/_2$×											
		$^1/_2$		$^7/_{16}$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$[c,f]	
Wt/ft		8.53		7.56		6.56		5.54		4.49		3.41	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_z	0	**16.7**	25.1	**15.7**	23.6	**14.5**	21.7	**13.1**	19.6	**11.4**	17.1	**8.81**	13.2
	1	**16.4**	24.7	**15.4**	23.1	**14.2**	21.3	**12.8**	19.2	**11.0**	16.6	**8.53**	12.8
	2	**15.5**	23.3	**14.5**	21.8	**13.3**	20.0	**11.9**	17.9	**10.2**	15.3	**7.87**	11.8
	3	**14.1**	21.3	**13.1**	19.7	**12.0**	18.0	**10.7**	16.0	**9.10**	13.7	**7.03**	10.6
	4	**12.5**	18.7	**11.5**	17.3	**10.5**	15.7	**9.27**	13.9	**7.88**	11.8	**6.09**	9.15
	5	**10.7**	16.0	**9.83**	14.8	**8.89**	13.4	**7.84**	11.8	**6.63**	9.97	**5.14**	7.72
	6	**8.90**	13.4	**8.15**	12.3	**7.34**	11.0	**6.44**	9.68	**5.42**	8.15	**4.23**	6.36
	7	**7.39**	11.1	**6.74**	10.1	**6.04**	9.08	**5.27**	7.93	**4.42**	6.64	**3.42**	5.14
	8	**6.20**	9.32	**5.64**	8.48	**5.04**	7.57	**4.38**	6.58	**3.65**	5.49	**2.81**	4.23

Properties

	$^1/_2$	$^7/_{16}$	$^3/_8$	$^5/_{16}$	$^1/_4$	$^3/_{16}$
A_g (in.^{2})	2.51	2.22	1.93	1.63	1.32	1.00
r_z (in.)	0.516	0.516	0.517	0.518	0.520	0.521

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

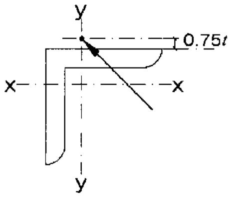
[c] Shape is slender for compression with F_y = 36 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L3–L2½

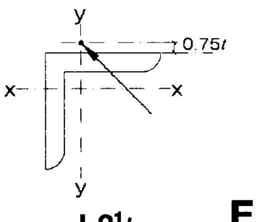
Shape		L3×2×										L2½×2½×	
		½		⅜		5/16		¼		3/16 c,f		½	
Wt/ft		7.70		5.95		5.03		4.09		3.12		7.65	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	13.0	19.6	11.6	17.4	10.5	15.8	9.25	13.9	7.29	11.0	14.5	21.8
	1	12.7	19.1	11.2	16.8	10.2	15.3	8.91	13.4	7.02	10.6	14.2	21.4
	2	11.7	17.6	10.2	15.3	9.23	13.9	8.05	12.1	6.35	9.54	13.5	20.2
	3	10.3	15.5	8.86	13.3	7.98	12.0	6.92	10.4	5.45	8.20	12.3	18.5
	4	8.70	13.1	7.41	11.1	6.63	9.97	5.71	8.59	4.51	6.77	10.8	16.3
	5	7.10	10.7	5.97	8.97	5.31	7.97	4.54	6.82	3.61	5.42	9.20	13.8
	6	5.76	8.65	4.78	7.19	4.22	6.35	3.59	5.39	2.83	4.26	7.54	11.3
	7	4.74	7.12	3.90	5.86	3.42	5.15	2.89	4.34	2.28	3.42	6.22	9.34
	8											5.19	7.80

Properties

A_g (in.^{2})	2.26	1.75	1.48	1.20	0.917	2.25
r_z (in.)	0.425	0.426	0.428	0.431	0.435	0.481

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.
[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.
Note: Heavy line indicates Kl/r equal to or greater than 200.



L2½

Table 4-12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L2½×2½×								L2½×2×	
		3/8		5/16		1/4		3/16^c		3/8	
Wt/ft		5.90		4.98		4.04		3.06		5.30	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_z	**0**	**12.5**	18.7	**11.2**	16.8	**9.70**	14.6	**7.88**	11.8	**10.7**	16.1
	1	**12.2**	18.3	**11.0**	16.5	**9.50**	14.3	**7.71**	11.6	**10.4**	15.6
	2	**11.5**	17.3	**10.3**	15.5	**8.93**	13.4	**7.19**	10.8	**9.49**	14.3
	3	**10.4**	15.7	**9.34**	14.0	**7.96**	12.0	**6.32**	9.50	**8.21**	12.3
	4	**9.10**	13.7	**8.03**	12.1	**6.81**	10.2	**5.38**	8.09	**6.81**	10.2
	5	**7.60**	11.4	**6.68**	10.0	**5.64**	8.47	**4.43**	6.66	**5.42**	8.15
	6	**6.16**	9.26	**5.38**	8.09	**4.52**	6.79	**3.53**	5.31	**4.32**	6.49
	7	**5.04**	7.57	**4.38**	6.58	**3.66**	5.49	**2.85**	4.28		
	8	**4.18**	6.28	**3.62**	5.44	**3.01**	4.52	**2.33**	3.50		

Properties

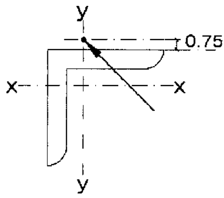
	3/8	5/16	1/4	3/16^c	3/8
A_g (in.^{2})	1.73	1.46	1.19	0.901	1.56
r_z (in.)	0.481	0.481	0.482	0.482	0.419

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.
Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 36 ksi

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles



L2½

Shape		L2½×2×						L2½×1½×			
		5/16		1/4		3/16^c		1/4		3/16^c	
Wt/ft		4.49		3.65		2.78		3.22		2.47	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	9.78	14.7	8.66	13.0	7.13	10.7	6.42	9.65	5.53	8.31
	1	9.49	14.3	8.33	12.5	6.81	10.2	6.07	9.12	5.18	7.79
	2	8.58	12.9	7.49	11.3	6.09	9.15	5.16	7.76	4.36	6.55
	3	7.38	11.1	6.40	9.62	5.18	7.78	4.08	6.14	3.40	5.12
	4	6.08	9.14	5.24	7.88	4.22	6.35	3.04	4.56	2.50	3.76
	5	4.81	7.23	4.12	6.19	3.30	4.97	2.28	3.43	1.85	2.78
	6	3.81	5.72	3.24	4.87	2.58	3.87				
	7			2.60	3.90	2.05	3.09				

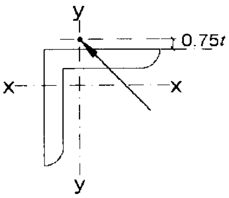
Properties

	L2½×2× 5/16	L2½×2× 1/4	L2½×2× 3/16^c	L2½×1½× 1/4	L2½×1½× 3/16^c
A_g (in.^{2})	1.32	1.07	0.818	0.947	0.724
r_z (in.)	0.420	0.423	0.426	0.321	0.324

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

^{c} Shape is slender for compression with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



L2

Table 4–12 (continued)
Available Strength in Axial Compression, kips
Eccentrically Loaded Single Angles

F_y = 36 ksi

Shape		L2×2×									
		3/8		5/16		1/4		3/16		1/8[c,f]	
Wt/ft		4.65		3.94		3.21		2.46		1.67	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_z	0	9.04	13.6	8.22	12.4	7.24	10.9	6.04	9.08	4.29	6.45
	1	8.78	13.2	7.98	12.0	7.02	10.5	5.85	8.79	4.16	6.26
	2	8.04	12.1	7.29	11.0	6.39	9.61	5.28	7.93	3.70	5.56
	3	6.96	10.5	6.28	9.43	5.43	8.16	4.41	6.63	3.09	4.64
	4	5.70	8.57	5.06	7.61	4.34	6.52	3.50	5.26	2.46	3.70
	5	4.43	6.66	3.90	5.87	3.32	4.99	2.66	4.00	1.87	2.81
	6	3.49	5.25	3.06	4.60	2.59	3.89	2.06	3.09	1.43	2.16

Properties					
A_g (in.^{2})	1.37	1.16	0.944	0.722	0.491
r_z (in.)	0.386	0.386	0.387	0.389	0.391

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

[c] Shape is slender for compression with F_y = 36 ksi.

[f] Shape exceeds compact limit for flexure with F_y = 36 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

4

COMPOSITE
HSS20–HSS16

Shape		HSS20×12×						HSS16×12×					
		5/8		1/2		3/8		5/8		1/2		3/8	
t_{design}, in.		0.581		0.465		0.349		0.581		0.465		0.349	
Steel Wt/ft		127		103		78.4		110		89.6		68.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	1150	1730	1010	1520	866	1300	970	1460	850	1270	725	1090
	6	1130	1700	994	1490	851	1280	954	1430	835	1250	712	1070
	7	1130	1690	988	1480	846	1270	948	1420	830	1240	708	1060
	8	1120	1680	981	1470	840	1260	941	1410	824	1240	703	1050
	9	1110	1660	973	1460	833	1250	934	1400	817	1230	697	1050
	10	1100	1650	965	1450	826	1240	925	1390	810	1220	690	1040
	11	1090	1630	955	1430	817	1230	916	1370	802	1200	683	1030
	12	1080	1620	945	1420	808	1210	906	1360	793	1190	676	1010
	13	1070	1600	934	1400	799	1200	896	1340	784	1180	668	1000
	14	1050	1580	923	1380	789	1180	885	1330	774	1160	659	988
	15	1040	1560	910	1370	778	1170	873	1310	764	1150	650	975
	16	1020	1540	898	1350	766	1150	860	1290	752	1130	640	960
	17	1010	1510	884	1330	754	1130	847	1270	741	1110	630	945
	18	993	1490	870	1300	742	1110	833	1250	729	1090	619	929
	19	976	1460	855	1280	729	1090	818	1230	716	1070	608	912
	20	959	1440	840	1260	715	1070	803	1210	703	1050	597	895
	21	941	1410	824	1240	702	1050	788	1180	689	1030	585	877
	22	923	1380	808	1210	687	1030	772	1160	675	1010	573	859
	23	904	1360	791	1190	673	1010	756	1130	661	992	560	840
	24	885	1330	774	1160	658	987	739	1110	646	970	548	821
	25	865	1300	757	1130	643	964	723	1080	632	947	535	802
	26	845	1270	739	1110	627	941	705	1060	616	925	522	782
	27	825	1240	721	1080	611	917	688	1030	601	902	508	762
	28	805	1210	703	1050	596	893	670	1010	586	879	495	742
	29	784	1180	685	1030	580	869	653	979	570	855	481	722
	30	763	1140	666	999	563	845	635	952	554	832	468	702
	32	721	1080	629	944	531	797	599	898	523	784	440	660
	34	679	1020	592	888	499	748	563	844	491	736	413	619
	36	637	955	555	832	466	700	527	790	459	689	386	579
	38	595	893	518	777	435	652	491	737	428	642	359	538
	40	554	832	482	723	403	605	456	684	398	597	333	499
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	528	794	432	649	331	497	379	569	310	466	239	359
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	372	559	303	455	234	352	310	466	255	383	196	295
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	72000		62600		52400		40200		35100		29200	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	30400		26400		21900		24800		21600		18000	
r_{mx}/r_{my}		1.54		1.54		1.55		1.27		1.28		1.27	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

4

COMPOSITE
HSS16–HSS14

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS16×12×		HSS16×8×								HSS14×10×	
		$^5/_{16}$		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^5/_8$	
t_{design}, in.		0.291		0.581		0.465		0.349		0.291		0.581	
Steel Wt/ft		57.4		93.1		75.9		58.1		48.9		93.1	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	661	992	762	1140	661	992	557	835	503	754	783	1170
	6	649	974	735	1100	638	957	537	805	484	727	764	1150
	7	645	968	725	1090	630	945	530	795	478	717	757	1140
	8	640	961	714	1070	620	930	522	783	471	706	749	1120
	9	635	953	702	1050	610	915	513	769	463	694	741	1110
	10	629	944	689	1030	598	897	503	754	454	680	731	1100
	11	622	934	674	1010	586	879	492	739	444	666	721	1080
	12	615	923	658	988	572	858	481	722	434	650	710	1060
	13	608	912	642	963	558	837	469	703	422	634	698	1050
	14	600	899	624	937	543	815	456	684	411	616	685	1030
	15	591	887	606	909	528	791	443	665	399	598	672	1010
	16	582	873	587	881	511	767	429	644	386	579	658	986
	17	573	859	568	852	495	742	415	623	373	560	643	965
	18	563	844	548	822	478	716	401	601	360	540	628	942
	19	552	829	528	792	460	690	386	579	347	520	612	918
	20	542	813	507	761	442	664	371	557	333	500	596	894
	21	531	796	487	730	425	637	356	534	319	479	580	870
	22	520	779	466	699	407	610	341	511	306	458	563	845
	23	508	762	445	667	389	583	326	488	292	438	546	819
	24	496	744	424	636	371	556	310	466	278	417	529	793
	25	484	726	404	605	353	529	295	443	264	396	512	767
	26	472	708	383	575	335	503	280	421	251	376	494	741
	27	460	690	363	544	318	477	266	399	237	356	477	715
	28	447	671	343	515	301	451	251	377	224	337	459	689
	29	435	652	324	486	284	426	237	356	212	317	442	663
	30	422	633	305	457	268	401	223	335	199	299	424	636
	32	397	596	268	403	236	354	197	295	175	263	390	585
	34	372	558	238	357	209	313	174	261	155	233	356	535
	36	347	520	212	318	186	280	155	233	138	208	324	486
	38	322	483	190	286	167	251	140	209	124	186	292	439
	40	298	447	172	258	151	226	126	189	112	168	264	396
Properties													
M_{nx}/Ω_b	ϕ_bM_{nx} kip-ft	201	303	296	445	243	366	188	283	159	240	275	414
M_{ny}/Ω_b	ϕ_bM_{ny} kip-ft	166	249	182	273	150	226	117	175	98.8	148	218	328
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	26200		28900		25500		21500		19200		24300	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	16000		9030		7930		6620		5890		13900	
r_{mx}/r_{my}		1.28		1.79		1.79		1.80		1.80		1.32	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 46 ksi
f'_c = 4 ksi

Table 4-13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

4

COMPOSITE
HSS14–HSS12

Shape		HSS14×10×								HSS12×10×			
		1/2		3/8		5/16		1/4		1/2		3/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.465		0.349	
Steel Wt/ft		75.9		58.1		48.9		39.5		69.1		52.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length $(KL)_y$ with respect to weak axis (ft)	0	**682**	1020	**577**	866	**523**	785	**468**	702	**608**	912	**513**	770
	6	**666**	998	**563**	845	**510**	765	**456**	685	**593**	890	**501**	751
	7	**660**	990	**558**	837	**506**	758	**452**	678	**588**	882	**496**	744
	8	**653**	980	**552**	828	**500**	750	**447**	671	**582**	873	**491**	736
	9	**646**	969	**546**	819	**494**	742	**442**	663	**575**	863	**485**	728
	10	**638**	956	**539**	808	**488**	732	**436**	654	**568**	851	**479**	718
	11	**629**	943	**531**	797	**481**	721	**429**	644	**559**	839	**472**	708
	12	**619**	928	**523**	784	**473**	710	**422**	633	**551**	826	**464**	696
	13	**609**	913	**514**	771	**465**	697	**415**	622	**541**	812	**456**	684
	14	**598**	897	**505**	757	**456**	684	**407**	610	**531**	797	**448**	672
	15	**586**	879	**495**	742	**447**	671	**398**	598	**521**	781	**439**	658
	16	**574**	861	**484**	726	**437**	656	**390**	584	**510**	765	**429**	644
	17	**562**	842	**473**	710	**428**	641	**380**	571	**498**	747	**420**	629
	18	**548**	823	**462**	693	**417**	626	**371**	557	**486**	729	**410**	614
	19	**535**	802	**451**	676	**407**	610	**361**	542	**474**	711	**399**	599
	20	**521**	782	**439**	658	**396**	593	**351**	527	**461**	692	**388**	583
	21	**507**	760	**427**	640	**384**	577	**341**	512	**449**	673	**377**	566
	22	**493**	739	**414**	621	**373**	560	**331**	496	**435**	653	**366**	549
	23	**478**	717	**402**	603	**362**	542	**320**	480	**422**	633	**355**	532
	24	**463**	695	**389**	584	**350**	525	**310**	464	**409**	613	**344**	515
	25	**448**	672	**376**	564	**338**	507	**299**	448	**395**	592	**332**	498
	26	**433**	649	**363**	545	**326**	489	**288**	432	**381**	572	**320**	481
	27	**418**	627	**350**	526	**314**	472	**277**	416	**368**	551	**309**	463
	28	**403**	604	**337**	506	**303**	454	**267**	400	**354**	531	**297**	446
	29	**387**	581	**324**	487	**291**	436	**256**	384	**340**	510	**286**	428
	30	**372**	559	**312**	467	**279**	418	**245**	368	**327**	490	**274**	411
	32	**343**	514	**286**	429	**256**	384	**224**	337	**300**	450	**251**	377
	34	**314**	470	**262**	392	**233**	350	**204**	306	**274**	411	**229**	344
	36	**285**	428	**238**	356	**212**	317	**185**	277	**248**	373	**208**	312
	38	**258**	387	**214**	321	**190**	286	**166**	248	**224**	336	**187**	281
	40	**233**	349	**193**	290	**172**	258	**149**	224	**202**	303	**169**	253

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	**227**	341	**175**	263	**148**	223	**120**	181	**181**	272	**140**	211
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	**180**	271	**139**	209	**118**	177	**95.8**	144	**160**	240	**124**	186
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	21600		18000		16000		14000		14400		12100	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	12200		10200		9030		7850		10600		8870	
r_{mx}/r_{my}		1.33		1.33		1.33		1.33		1.16		1.17	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

4

COMPOSITE HSS12

Table 4-13 (continued)
Available Strength in Axial Compression, kips

F_y = 46 ksi
f'_c = 4 ksi

Concrete Filled Rectangular HSS

Shape		HSS12×10×				HSS12×8×							
		5/16		1/4		5/8		1/2		3/8		1/4	
t_{design}, in.		0.291		0.233		0.581		0.465		0.349		0.233	
Steel Wt/ft		44.6		36.0		76.1		62.3		47.8		32.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	464	697	414	622	609	913	528	792	443	664	354	531
	6	453	679	404	606	586	879	508	762	426	640	340	510
	7	449	673	400	600	578	867	501	752	421	631	335	503
	8	444	666	396	593	569	853	494	740	414	621	330	495
	9	438	658	391	586	558	838	485	727	407	610	324	486
	10	433	649	385	578	547	821	475	713	399	598	317	476
	11	426	639	380	569	535	803	465	698	390	585	310	465
	12	419	629	373	560	522	783	454	681	381	571	303	454
	13	412	618	367	550	508	762	442	663	371	556	295	442
	14	404	606	359	539	494	741	430	645	361	541	286	429
	15	396	594	352	528	479	718	417	626	350	525	277	416
	16	387	581	344	516	463	695	404	606	339	508	268	402
	17	378	568	336	504	447	671	390	585	327	491	259	388
	18	369	554	328	491	431	646	376	564	315	473	249	374
	19	360	540	319	478	414	621	362	543	303	455	239	359
	20	350	525	310	465	397	596	347	521	291	437	229	344
	21	340	510	301	451	380	570	333	499	279	418	219	329
	22	330	495	292	437	363	544	318	477	267	400	209	314
	23	319	479	282	423	346	519	303	455	254	382	199	299
	24	309	463	273	409	329	493	289	433	242	363	189	284
	25	298	448	263	395	312	468	274	412	230	345	180	270
	26	288	432	254	380	296	443	260	390	218	327	170	255
	27	277	416	244	366	279	419	246	369	206	309	161	241
	28	267	400	235	352	263	395	232	348	195	292	151	227
	29	256	384	225	338	248	372	219	328	183	275	142	213
	30	246	368	216	323	232	348	206	308	172	259	133	200
	32	225	337	197	295	204	306	181	271	152	227	117	176
	34	205	307	179	268	181	271	160	240	134	201	104	155
	36	186	278	162	243	161	242	143	214	120	180	92.4	139
	38	167	250	145	218	145	217	128	192	107	161	83.0	124
	40	151	226	131	196	131	196	116	173	97.0	145	74.9	112
Properties													
M_{nx}/Ω_b $\phi_b M_{nx}$	kip-ft	119	179	96.6	145	188	283	156	235	122	183	84.0	126
M_{ny}/Ω_b $\phi_b M_{ny}$	kip-ft	105	158	85.4	128	142	214	118	178	92.1	138	63.8	95.8
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	10800		9370		13500		11900		10100		7860	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	7910		6880		6870		6080		5100		3930	
r_{mx}/r_{my}		1.17		1.17		1.40		1.40		1.41		1.41	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

4
COMPOSITE
HSS12–HSS10

Shape		HSS12×6×								HSS10×8×			
		5/8		1/2		3/8		1/4		5/8		1/2	
t_{design}, in.		0.581		0.465		0.349		0.233		0.581		0.465	
Steel Wt/ft		67.6		55.5		42.7		29.2		67.6		55.5	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	519	778	447	671	372	558	293	440	532	798	461	692
	6	485	728	419	629	349	524	275	412	511	767	443	665
	7	473	710	409	614	341	512	268	402	504	756	437	656
	8	460	691	398	597	332	498	261	391	496	744	430	645
	9	446	669	386	579	322	483	253	379	487	730	422	633
	10	431	646	373	560	311	467	244	367	476	715	414	620
	11	414	621	359	539	300	450	235	353	465	698	404	606
	12	397	595	344	517	288	432	226	338	454	681	394	591
	13	379	568	329	494	275	413	216	323	441	662	384	576
	14	360	540	313	470	262	394	205	308	428	642	373	559
	15	341	512	297	446	249	374	195	292	415	622	361	542
	16	322	483	281	421	236	354	184	276	401	601	349	524
	17	303	454	265	397	222	333	173	260	386	579	337	505
	18	284	426	248	372	209	313	163	244	372	557	324	487
	19	265	397	232	348	195	293	152	228	357	535	312	467
	20	246	370	216	324	182	273	142	212	342	512	299	448
	21	228	343	201	301	169	254	131	197	326	490	286	429
	22	211	316	186	278	157	235	122	182	311	467	273	409
	23	193	290	171	256	145	217	112	168	296	444	260	389
	24	178	267	157	235	133	199	103	154	281	421	247	370
	25	164	246	145	217	122	184	94.7	142	266	399	234	351
	26	151	227	134	200	113	170	87.5	131	251	377	221	332
	27	140	211	124	186	105	158	81.2	122	237	356	209	314
	28	131	196	115	173	97.6	146	75.5	113	223	335	197	295
	29	122	183	107	161	91.0	137	70.4	106	209	314	185	278
	30	114	171	100	151	85.1	128	65.7	98.6	196	294	173	260
	32	99.9	150	88.2	132	74.8	112	57.8	86.7	172	258	152	228
	34	88.5	133	78.1	117	66.2	99.3	51.2	76.8	152	229	135	202
	36	79.0	118	69.7	105	59.1	88.6	45.7	68.5	136	204	120	180
	38	70.9	106	62.5	93.8	53.0	79.5	41.0	61.5	122	183	108	162
	40			56.5	84.7	47.8	71.8	37.0	55.5	110	165	97.5	146
Properties													
M_{nx}/Ω_b ϕ_bM_{nx}	kip-ft	158	237	132	198	103	155	71.4	107	143	215	119	179
M_{ny}/Ω_b ϕ_bM_{ny}	kip-ft	96.6	145	80.9	122	63.5	95.4	44.3	66.5	122	184	102	153
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	10700		9460		8090		6310		8400		7440	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	3360		2970		2520		1940		5790		5130	
r_{mx}/r_{my}		1.78		1.79		1.79		1.80		1.20		1.21	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

4

COMPOSITE HSS10

Table 4-13 (continued)
Available Strength in Axial Compression, kips

F_y = 46 ksi
f'_c = 4 ksi

Concrete Filled Rectangular HSS

Shape		HSS10×8×								HSS10×6×			
		3/8		5/16		1/4		3/16		5/8		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.581		0.465	
Steel Wt/ft		42.7		36.1		29.2		22.2		59.1		48.7	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	386	579	347	520	307	460	265	398	449	673	387	581
	6	371	557	334	500	295	442	254	381	419	628	362	543
	7	366	549	329	493	291	436	251	376	408	612	353	530
	8	360	540	324	485	286	429	246	369	397	595	344	515
	9	354	531	318	477	281	421	242	362	384	576	333	499
	10	347	520	311	467	275	412	236	355	370	555	321	482
	11	339	508	304	456	268	403	231	346	355	533	309	463
	12	331	496	297	445	262	393	225	337	340	510	296	444
	13	322	483	289	433	255	382	218	328	324	486	282	423
	14	313	469	281	421	247	371	212	318	307	461	268	402
	15	303	455	272	408	239	359	205	307	291	436	254	381
	16	293	440	263	395	231	347	198	297	274	410	240	360
	17	283	425	254	381	223	335	190	286	257	385	225	338
	18	273	409	244	367	215	322	183	274	240	360	211	316
	19	262	393	235	352	206	309	175	263	223	335	197	295
	20	252	377	225	338	197	296	168	252	207	311	183	274
	21	241	361	215	323	189	283	160	240	191	287	169	254
	22	230	345	206	309	180	270	152	228	176	264	156	234
	23	219	329	196	294	171	257	145	217	161	242	143	215
	24	208	313	186	279	163	244	137	206	148	222	132	197
	25	198	297	177	265	154	231	130	194	136	205	121	182
	26	187	281	167	251	146	218	122	183	126	189	112	168
	27	177	265	158	237	137	206	115	173	117	175	104	156
	28	167	250	149	223	129	194	108	162	109	163	96.7	145
	29	157	235	140	210	121	182	101	152	101	152	90.2	135
	30	147	221	131	197	114	170	94.5	142	94.7	142	84.2	126
	32	129	194	115	173	99.8	150	83.0	125	83.3	125	74.0	111
	34	115	172	102	153	88.4	133	73.5	110	73.8	111	65.6	98.4
	36	102	153	91.0	136	78.8	118	65.6	98.4	65.8	98.7	58.5	87.8
	38	91.7	138	81.7	123	70.8	106	58.9	88.3	59.0	88.6	52.5	78.8
	40	82.8	124	73.7	111	63.9	95.8	53.1	79.7				
Properties													
M_{nx}/Ω_b ϕ_bM_{nx}	kip-ft	93.0	140	79.0	119	64.5	97.0	49.1	73.8	118	177	98.7	148
M_{ny}/Ω_b ϕ_bM_{ny}	kip-ft	79.8	120	67.9	102	55.4	83.3	42.2	63.5	82.1	123	69.1	104
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	6320		5640		4900		4090		6550		5820	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	4350		3870		3360		2790		2790		2490	
r_{mx}/r_{my}		1.21		1.21		1.21		1.21		1.53		1.53	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

4
COMPOSITE HSS10

Shape		HSS10×6×								HSS10×5×			
		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^3/_8$		$^5/_{16}$	
t_{design}, in.		0.349		0.291		0.233		0.174		0.349		0.291	
Steel Wt/ft		37.6		31.8		25.8		19.7		35.1		29.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	322	483	288	432	253	379	216	324	290	435	259	388
	6	302	452	270	405	237	355	202	303	264	397	236	354
	7	294	442	263	395	231	346	197	295	256	383	228	342
	8	286	430	256	384	225	337	191	287	246	369	219	329
	9	278	416	248	373	218	327	185	278	235	353	210	315
	10	268	402	240	360	210	315	179	268	224	336	200	300
	11	258	387	231	346	202	303	172	257	212	318	189	284
	12	247	371	221	332	194	291	164	246	200	300	179	268
	13	236	354	212	317	185	277	157	235	187	281	167	251
	14	225	337	201	302	176	264	149	223	174	262	156	234
	15	213	320	191	287	167	250	141	211	162	243	145	218
	16	201	302	181	271	157	236	133	199	149	224	134	201
	17	190	284	170	255	148	222	124	187	137	206	123	185
	18	178	267	159	239	139	208	116	175	125	188	112	169
	19	166	249	149	224	130	194	108	163	114	171	102	153
	20	155	232	139	208	121	181	101	151	103	154	92.3	138
	21	143	215	129	193	112	168	93.1	140	93.1	140	83.7	126
	22	133	199	119	179	103	155	85.7	129	84.8	127	76.3	114
	23	122	183	110	164	94.8	142	78.5	118	77.6	116	69.8	105
	24	112	168	101	151	87.1	131	72.1	108	71.3	107	64.1	96.2
	25	103	155	92.7	139	80.3	120	66.4	99.6	65.7	98.5	59.1	88.6
	26	95.4	143	85.7	129	74.2	111	61.4	92.1	60.7	91.1	54.6	81.9
	27	88.4	133	79.5	119	68.8	103	56.9	85.4	56.3	84.4	50.7	76.0
	28	82.2	123	73.9	111	64.0	96.0	53.0	79.4	52.3	78.5	47.1	70.7
	29	76.7	115	68.9	103	59.6	89.5	49.4	74.0	48.8	73.2	43.9	65.9
	30	71.6	107	64.4	96.6	55.7	83.6	46.1	69.2	45.6	68.4	41.0	61.5
	32	63.0	94.4	56.6	84.9	49.0	73.5	40.5	60.8	40.1	60.1	36.1	54.1
	34	55.8	83.7	50.1	75.2	43.4	65.1	35.9	53.9	35.5	53.3	31.9	47.9
	36	49.7	74.6	44.7	67.1	38.7	58.1	32.0	48.0				
	38	44.7	67.0	40.1	60.2	34.7	52.1	28.7	43.1				
	40	40.3	60.4	36.2	54.3	31.4	47.0	25.9	38.9				
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	77.5	116	66.1	99.3	54.1	81.3	41.3	62.0	69.8	105	59.6	89.5
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	54.4	81.8	46.5	69.8	38.1	57.2	29.1	43.8	42.9	64.5	36.7	55.2
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	5000		4520		3920		3270		4310		3900	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	2120		1900		1640		1360		1350		1210	
r_{mx}/r_{my}		1.54		1.54		1.54		1.55		1.79		1.79	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

4

COMPOSITE
HSS10–HSS9

Table 4-13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS10×5×				HSS9×7×							
		1/4		3/16		5/8		1/2		3/8		5/16	
t_{design}, in.		0.233		0.174		0.581		0.465		0.349		0.291	
Steel Wt/ft		24.1		18.4		59.1		48.7		37.6		31.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	226	339	192	288	454	681	392	589	327	491	293	440
	6	206	309	174	261	430	646	373	559	311	467	279	418
	7	199	299	168	253	422	633	366	549	306	458	274	411
	8	192	287	162	243	413	620	358	537	299	449	268	402
	9	183	275	155	232	403	604	350	524	292	438	262	393
	10	175	262	147	221	392	588	340	510	285	427	255	382
	11	165	248	139	209	380	570	330	495	276	415	248	371
	12	156	234	131	196	367	551	319	479	268	401	240	360
	13	146	219	123	184	354	531	308	462	258	388	232	347
	14	136	204	114	171	340	510	297	445	249	373	223	334
	15	126	190	106	158	326	489	284	427	239	359	214	321
	16	117	175	97.3	146	312	467	272	408	229	343	205	308
	17	107	161	89.1	134	297	445	260	389	219	328	196	294
	18	98.0	147	81.2	122	282	423	247	370	208	312	187	280
	19	89.1	134	73.6	110	267	401	234	351	198	297	177	266
	20	80.4	121	66.4	99.6	252	378	221	332	187	281	168	252
	21	72.9	109	60.2	90.3	237	356	209	313	177	265	158	238
	22	66.4	99.7	54.9	82.3	223	334	196	295	166	250	149	224
	23	60.8	91.2	50.2	75.3	209	313	184	276	156	235	140	210
	24	55.8	83.7	46.1	69.2	195	292	172	258	146	220	131	197
	25	51.4	77.2	42.5	63.7	181	272	161	241	137	205	123	184
	26	47.6	71.4	39.3	58.9	168	252	149	223	127	191	114	171
	27	44.1	66.2	36.4	54.6	156	233	138	207	118	177	106	159
	28	41.0	61.5	33.9	50.8	145	217	128	193	110	165	98.4	148
	29	38.2	57.4	31.6	47.4	135	202	120	180	102	153	91.7	138
	30	35.7	53.6	29.5	44.3	126	189	112	168	95.6	143	85.7	129
	32	31.4	47.1	25.9	38.9	111	166	98.4	148	84.0	126	75.3	113
	34	27.8	41.7	23.0	34.5	98.1	147	87.1	131	74.4	112	66.7	100
	36					87.5	131	77.7	117	66.4	99.6	59.5	89.3
	38					78.5	118	69.8	105	59.6	89.4	53.4	80.1
	40					70.9	106	63.0	94.4	53.8	80.7	48.2	72.3
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	48.8	73.4	37.3	56.1	111	167	92.9	140	72.9	110	62.2	93.4
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	30.2	45.4	23.2	34.8	93.0	140	78.1	117	61.4	92.3	52.4	78.7
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	3410		2850		5640		5050		4320		3870	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	1060		873		3730		3310		2820		2530	
r_{mx}/r_{my}		1.80		1.81		1.23		1.24		1.24		1.24	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

4

COMPOSITE HSS9

Shape		HSS9×5×											
		5/8		1/2		3/8		5/16		1/4		3/16	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		50.6		41.9		32.5		27.6		22.4		17.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	374	561	322	483	267	400	238	357	208	311	176	264
	6	338	506	292	438	243	364	216	325	189	283	160	240
	7	325	488	282	423	234	352	209	314	183	274	154	231
	8	312	468	271	406	225	338	201	302	176	263	148	222
	9	297	446	258	388	216	323	192	289	168	252	142	212
	10	282	422	245	368	205	307	183	275	160	240	135	202
	11	265	398	232	348	194	291	173	260	151	227	127	191
	12	249	373	218	327	182	274	163	245	143	214	120	179
	13	232	347	203	305	171	256	153	229	134	200	112	168
	14	215	322	189	283	159	239	143	214	124	187	104	156
	15	198	296	175	262	147	221	132	198	115	173	96.3	145
	16	181	272	160	241	136	204	122	183	106	160	88.7	133
	17	165	247	147	220	124	187	112	168	97.6	146	81.2	122
	18	149	224	133	200	113	170	102	153	89.1	134	73.9	111
	19	134	201	120	180	103	154	92.6	139	80.9	121	66.9	100
	20	121	182	109	163	92.8	139	83.6	125	73.0	110	60.3	90.5
	21	110	165	98.5	148	84.2	126	75.8	114	66.2	99.3	54.7	82.1
	22	100	150	89.7	135	76.7	115	69.1	104	60.3	90.5	49.9	74.8
	23	91.6	137	82.1	123	70.2	105	63.2	94.8	55.2	82.8	45.6	68.4
	24	84.1	126	75.4	113	64.4	96.7	58.0	87.1	50.7	76.0	41.9	62.9
	25	77.5	116	69.5	104	59.4	89.1	53.5	80.2	46.7	70.1	38.6	57.9
	26	71.7	107	64.2	96.4	54.9	82.4	49.5	74.2	43.2	64.8	35.7	53.6
	27	66.4	99.7	59.6	89.4	50.9	76.4	45.9	68.8	40.1	60.1	33.1	49.7
	28	61.8	92.7	55.4	83.1	47.3	71.0	42.6	64.0	37.2	55.9	30.8	46.2
	29	57.6	86.4	51.6	77.5	44.1	66.2	39.8	59.6	34.7	52.1	28.7	43.1
	30	53.8	80.7	48.3	72.4	41.2	61.9	37.1	55.7	32.4	48.7	26.8	40.2
	32	47.3	71.0	42.4	63.6	36.2	54.4	32.6	49.0	28.5	42.8	23.6	35.4
	34							28.9	43.4	25.3	37.9	20.9	31.3

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	88.3	133	74.7	112	59.1	88.8	50.5	75.9	41.5	62.4	31.8	47.8
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	58.1	87.3	49.3	74.1	39.2	58.9	33.6	50.5	27.7	41.6	21.2	31.9

$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	4230	3800	3250	2950	2590	2160
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	1580	1430	1220	1100	959	792
r_{mx}/r_{my}		1.63	1.63	1.63	1.64	1.64	1.65

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

4

COMPOSITE HSS8

Table 4–13 (continued)

Available Strength in Axial Compression, kips

Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS8×6×											
		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		50.6		41.9		32.5		27.6		22.4		17.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	379	568	327	491	272	408	243	364	213	319	181	271
	6	352	528	305	458	254	381	227	340	199	298	169	253
	7	343	515	297	446	248	372	221	332	194	291	164	247
	8	333	499	289	433	241	361	215	323	188	282	160	240
	9	321	482	279	419	233	350	208	313	182	274	155	232
	10	309	464	269	404	225	337	201	302	176	264	149	223
	11	296	445	258	387	216	324	193	290	169	254	143	214
	12	283	424	247	370	207	310	185	278	162	243	137	205
	13	269	403	235	353	197	296	177	265	154	231	130	195
	14	255	382	223	334	187	281	168	252	147	220	124	185
	15	240	360	211	316	177	266	159	238	139	208	117	175
	16	225	338	198	297	167	251	150	225	131	196	110	165
	17	211	316	186	279	157	235	141	211	123	184	103	155
	18	196	295	174	260	147	220	132	198	115	173	96.3	144
	19	182	273	161	242	137	205	123	184	107	161	89.6	134
	20	168	252	150	224	127	190	114	171	99.6	149	83.0	125
	21	155	232	138	207	117	176	106	159	92.1	138	76.6	115
	22	142	212	127	190	108	162	97.4	146	84.9	127	70.4	106
	23	130	194	116	174	99.1	149	89.3	134	77.8	117	64.5	96.7
	24	119	179	106	160	91.0	136	82.0	123	71.4	107	59.2	88.8
	25	110	165	98.1	147	83.8	126	75.6	113	65.8	98.8	54.6	81.8
	26	101	152	90.7	136	77.5	116	69.9	105	60.9	91.3	50.4	75.7
	27	94.0	141	84.1	126	71.9	108	64.8	97.2	56.4	84.7	46.8	70.2
	28	87.4	131	78.2	117	66.8	100	60.3	90.4	52.5	78.7	43.5	65.2
	29	81.5	122	72.9	109	62.3	93.5	56.2	84.3	48.9	73.4	40.5	60.8
	30	76.2	114	68.1	102	58.2	87.3	52.5	78.8	45.7	68.6	37.9	56.8
	32	66.9	100	59.9	89.8	51.2	76.8	46.1	69.2	40.2	60.3	33.3	49.9
	34	59.3	88.9	53.1	79.6	45.3	68.0	40.9	61.3	35.6	53.4	29.5	44.2
	36	52.9	79.3	47.3	71.0	40.4	60.7	36.5	54.7	31.8	47.6	26.3	39.5
	38			42.5	63.7	36.3	54.4	32.7	49.1	28.5	42.7	23.6	35.4
	40							29.5	44.3	25.7	38.6	21.3	32.0
Properties													
M_{nx}/Ω_b | $\phi_b M_{nx}$	kip-ft	82.8	124	69.9	105	55.3	83.1	47.3	71.1	38.8	58.4	29.7	44.7
M_{ny}/Ω_b | $\phi_b M_{ny}$	kip-ft	67.7	102	57.3	86.1	45.4	68.2	38.8	58.4	31.9	48.0	24.5	36.8
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	3630		3240		2780		2520		2190		1830	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	2250		2020		1730		1560		1350		1120	
r_{mx}/r_{my}			1.27		1.27		1.27		1.27		1.27		1.28

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

4

COMPOSITE HSS8

Shape		HSS8×4×											
		5/8		1/2		3/8		5/16		1/4		3/16	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		42.1		35.1		27.4		23.3		19.0		14.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	302	453	261	391	215	323	191	286	166	248	139	208
	6	256	385	223	334	185	278	165	247	143	215	120	180
	7	242	362	211	316	176	263	156	234	136	204	114	171
	8	226	338	197	296	165	247	147	221	128	192	107	161
	9	209	313	183	275	154	231	137	206	120	179	100	150
	10	191	287	169	253	142	213	127	191	111	166	92.7	139
	11	174	261	154	231	130	195	117	175	102	153	85.1	128
	12	157	235	140	209	118	178	106	159	92.9	139	77.5	116
	13	140	210	125	188	107	160	95.9	144	84.0	126	70.1	105
	14	123	185	111	167	95.5	143	85.9	129	75.4	113	62.8	94.2
	15	108	162	98.1	147	84.7	127	76.4	115	67.1	101	55.8	83.8
	16	94.9	142	86.2	129	74.4	112	67.2	101	59.1	88.7	49.2	73.8
	17	84.1	126	76.4	115	65.9	98.9	59.6	89.3	52.4	78.5	43.6	65.4
	18	75.0	112	68.1	102	58.8	88.2	53.1	79.7	46.7	70.1	38.9	58.3
	19	67.3	101	61.1	91.7	52.8	79.2	47.7	71.5	41.9	62.9	34.9	52.3
	20	60.7	91.1	55.2	82.7	47.6	71.4	43.0	64.5	37.8	56.7	31.5	47.2
	21	55.1	82.6	50.0	75.1	43.2	64.8	39.0	58.5	34.3	51.5	28.6	42.8
	22	50.2	75.3	45.6	68.4	39.4	59.0	35.6	53.3	31.3	46.9	26.0	39.0
	23	45.9	68.9	41.7	62.6	36.0	54.0	32.5	48.8	28.6	42.9	23.8	35.7
	24	42.2	63.3	38.3	57.5	33.1	49.6	29.9	44.8	26.3	39.4	21.9	32.8
	25	38.9	58.3	35.3	53.0	30.5	45.7	27.5	41.3	24.2	36.3	20.1	30.2
	26			32.6	49.0	28.2	42.3	25.5	38.2	22.4	33.6	18.6	27.9
	27							23.6	35.4	20.8	31.1	17.3	25.9
	28											16.1	24.1

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	63.0	94.7	53.8	80.9	43.0	64.7	37.0	55.6	30.5	45.9	23.5	35.3
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	38.1	57.2	32.8	49.3	26.4	39.6	22.7	34.2	18.8	28.3	14.5	21.8
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	2530		2300		1990		1810		1600		1340	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	798		724		625		566		497		413	
r_{mx}/r_{my}			1.78		1.78		1.78		1.79		1.79		1.80

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

4

COMPOSITE
HSS7

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS7×5×											
		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		35.1		27.4		23.3		19.0		14.5		9.85	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	266	398	220	330	196	294	171	256	144	216	117	175
	6	240	359	199	299	178	266	155	232	131	196	105	158
	7	231	346	192	288	171	257	150	224	126	189	101	152
	8	221	332	184	277	165	247	144	216	121	181	97.2	146
	9	211	316	176	264	157	236	137	206	115	173	92.5	139
	10	199	299	167	251	149	224	130	196	110	164	87.6	131
	11	188	282	158	236	141	211	123	185	103	155	82.5	124
	12	176	264	148	222	132	199	116	174	97.1	146	77.2	116
	13	164	245	138	207	124	186	108	162	90.6	136	71.8	108
	14	151	227	128	192	115	172	101	151	84.2	126	66.5	99.7
	15	139	209	118	177	106	159	93.2	140	77.8	117	61.1	91.7
	16	127	191	109	163	97.7	146	85.7	129	71.4	107	55.9	83.9
	17	116	174	99.1	149	89.3	134	78.4	118	65.2	97.9	50.9	76.3
	18	105	157	90.0	135	81.2	122	71.4	107	59.3	88.9	46.0	69.0
	19	94.2	141	81.1	122	73.3	110	64.5	96.8	53.5	80.2	41.3	61.9
	20	85.0	127	73.2	110	66.1	99.2	58.2	87.3	48.2	72.4	37.2	55.9
	21	77.1	116	66.4	99.6	60.0	89.9	52.8	79.2	43.8	65.6	33.8	50.7
	22	70.2	105	60.5	90.7	54.6	82.0	48.1	72.2	39.9	59.8	30.8	46.2
	23	64.3	96.4	55.3	83.0	50.0	75.0	44.0	66.0	36.5	54.7	28.2	42.2
	24	59.0	88.5	50.8	76.2	45.9	68.9	40.4	60.6	33.5	50.2	25.9	38.8
	25	54.4	81.6	46.8	70.3	42.3	63.5	37.3	55.9	30.9	46.3	23.8	35.8
	26	50.3	75.4	43.3	65.0	39.1	58.7	34.4	51.7	28.5	42.8	22.0	33.1
	27	46.6	70.0	40.2	60.2	36.3	54.4	31.9	47.9	26.5	39.7	20.4	30.7
	28	43.4	65.1	37.3	56.0	33.7	50.6	29.7	44.6	24.6	36.9	19.0	28.5
	29	40.4	60.6	34.8	52.2	31.4	47.2	27.7	41.5	22.9	34.4	17.7	26.6
	30	37.8	56.7	32.5	48.8	29.4	44.1	25.9	38.8	21.4	32.2	16.6	24.8
	32			28.6	42.9	25.8	38.7	22.7	34.1	18.8	28.3	14.6	21.8
	34									16.7	25.0	12.9	19.3

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	50.2	75.4	40.1	60.2	34.4	51.8	28.4	42.7	21.8	32.8	15.0	22.5
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	39.6	59.6	31.7	47.7	27.3	41.1	22.6	33.9	17.4	26.1	11.9	17.9
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	1940		1680		1530		1350		1120		871	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	1120		962		868		765		634		490	
r_{mx}/r_{my}			1.32		1.32		1.33		1.33		1.33		1.33	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

4
COMPOSITE HSS7

Shape		HSS7×4×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		31.7		24.9		21.2		17.3		13.3		9.00	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	234	351	193	290	172	257	149	223	125	187	100	150
	6	199	299	166	249	148	222	128	193	108	161	85.7	129
	7	188	282	157	236	140	210	122	183	102	153	81.1	122
	8	176	264	147	221	132	197	115	172	95.8	144	76.0	114
	9	163	245	137	206	123	184	107	160	89.4	134	70.7	106
	10	150	225	127	190	113	170	98.8	148	82.6	124	65.1	97.7
	11	137	205	116	174	104	156	90.7	136	75.8	114	59.5	89.3
	12	123	185	105	157	94.3	141	82.5	124	68.9	103	54.0	80.9
	13	110	165	94.4	142	85.0	127	74.5	112	62.2	93.3	48.5	72.7
	14	97.8	147	84.2	126	76.0	114	66.7	100	55.7	83.5	43.2	64.8
	15	85.7	129	74.4	112	67.3	101	59.2	88.9	49.4	74.1	38.1	57.1
	16	75.4	113	65.4	98.1	59.2	88.8	52.1	78.2	43.4	65.2	33.5	50.2
	17	66.8	100	57.9	86.9	52.4	78.6	46.2	69.3	38.5	57.7	29.6	44.5
	18	59.5	89.3	51.7	77.5	46.8	70.1	41.2	61.8	34.3	51.5	26.4	39.7
	19	53.4	80.2	46.4	69.5	42.0	63.0	37.0	55.4	30.8	46.2	23.7	35.6
	20	48.2	72.3	41.8	62.8	37.9	56.8	33.4	50.0	27.8	41.7	21.4	32.1
	21	43.7	65.6	37.9	56.9	34.4	51.5	30.3	45.4	25.2	37.8	19.4	29.1
	22	39.9	59.8	34.6	51.9	31.3	47.0	27.6	41.4	23.0	34.5	17.7	26.5
	23	36.5	54.7	31.6	47.5	28.6	43.0	25.2	37.8	21.0	31.5	16.2	24.3
	24	33.5	50.2	29.1	43.6	26.3	39.5	23.2	34.7	19.3	29.0	14.9	22.3
	25	30.9	46.3	26.8	40.2	24.2	36.4	21.3	32.0	17.8	26.7	13.7	20.6
	26			24.8	37.1	22.4	33.6	19.7	29.6	16.4	24.7	12.7	19.0
	27							18.3	27.5	15.3	22.9	11.8	17.6
	28											10.9	16.4

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	43.2	64.9	34.7	52.2	30.0	45.0	24.8	37.3	19.1	28.7	13.2	19.8
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	29.0	43.6	23.4	35.2	20.3	30.5	16.8	25.3	13.0	19.6	8.97	13.5
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	1610		1400		1270		1120		943		733	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	634		550		498		438		366		281	
r_{mx}/r_{my}			1.60		1.59		1.59		1.60		1.61		1.62	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

4

COMPOSITE
HSS6

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS6×5×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		31.7		24.9		21.2		17.3		13.3		9.00	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	237	356	197	295	175	263	152	228	128	192	103	155
	1	237	355	196	294	175	262	152	228	128	192	103	155
	2	235	352	195	292	173	260	151	226	127	190	102	153
	3	231	347	192	288	171	256	149	223	125	187	101	151
	4	226	339	188	282	167	251	146	219	123	184	98.7	148
	5	220	331	183	275	163	245	142	213	119	179	96.1	144
	6	213	320	178	266	158	237	138	207	116	174	93.1	140
	7	205	308	171	257	153	229	133	199	112	168	89.6	134
	8	196	294	164	246	146	219	128	191	107	161	85.8	129
	9	187	280	156	234	140	209	122	183	102	153	81.7	123
	10	176	264	148	222	132	198	116	173	97.0	145	77.3	116
	11	166	248	139	209	125	187	109	164	91.4	137	72.7	109
	12	155	232	131	196	117	175	102	153	85.8	129	68.0	102
	13	144	215	122	182	109	164	95.5	143	80.0	120	63.2	94.8
	14	132	199	113	169	101	152	88.6	133	74.2	111	58.4	87.6
	15	122	182	104	155	93.2	140	81.8	123	68.4	103	53.7	80.5
	16	111	166	94.9	142	85.5	128	75.1	113	62.8	94.2	49.1	73.6
	17	100	151	86.4	130	77.9	117	68.6	103	57.3	85.9	44.6	66.9
	18	90.4	136	78.2	117	70.7	106	62.2	93.4	51.9	77.9	40.2	60.3
	19	81.1	122	70.2	105	63.6	95.4	56.1	84.1	46.7	70.1	36.1	54.2
	20	73.2	110	63.4	95.1	57.4	86.1	50.6	75.9	42.2	63.3	32.6	48.9
	21	66.4	99.6	57.5	86.2	52.0	78.1	45.9	68.9	38.3	57.4	29.6	44.3
	22	60.5	90.8	52.4	78.6	47.4	71.1	41.8	62.7	34.9	52.3	26.9	40.4
	23	55.4	83.0	47.9	71.9	43.4	65.1	38.3	57.4	31.9	47.8	24.6	37.0
	24	50.8	76.3	44.0	66.0	39.8	59.8	35.1	52.7	29.3	43.9	22.6	33.9
	25	46.9	70.3	40.6	60.8	36.7	55.1	32.4	48.6	27.0	40.5	20.9	31.3
	26	43.3	65.0	37.5	56.2	33.9	50.9	29.9	44.9	25.0	37.4	19.3	28.9
	27	40.2	60.3	34.8	52.2	31.5	47.2	27.8	41.7	23.1	34.7	17.9	26.8
	28	37.4	56.0	32.3	48.5	29.3	43.9	25.8	38.7	21.5	32.3	16.6	24.9
	29	34.8	52.2	30.1	45.2	27.3	40.9	24.1	36.1	20.1	30.1	15.5	23.2
	30	32.5	48.8	28.2	42.2	25.5	38.3	22.5	33.7	18.7	28.1	14.5	21.7
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	39.5	59.4	31.8	47.8	27.4	41.2	22.7	34.1	17.5	26.3	12.0	18.1
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	34.8	52.3	28.0	42.1	24.2	36.3	20.0	30.1	15.4	23.2	10.6	16.0
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	1300		1130		1020		903		753		585	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	962		832		753		665		554		428	
r_{mx}/r_{my}		1.16		1.16		1.16		1.16		1.17		1.17	

ASD	LRFD
Ω_c = 2.00	ϕ_c = 0.75

F_y = 46 ksi
f'_c = 4 ksi

Table 4-13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

COMPOSITE HSS6

Shape		HSS6×4×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		28.3		22.3		19.1		15.6		12.0		8.15	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	207	311	172	258	152	229	132	198	111	166	88.3	132
	1	206	310	171	256	152	228	132	197	110	165	87.9	132
	2	204	305	169	253	150	225	130	195	109	163	86.7	130
	3	199	298	165	248	147	220	127	191	106	160	84.9	127
	4	193	289	160	240	142	213	124	185	103	155	82.3	123
	5	185	277	154	231	137	205	119	178	99.5	149	79.2	119
	6	176	263	147	220	131	196	114	170	95.1	143	75.5	113
	7	165	248	139	208	124	185	108	161	90.0	135	71.4	107
	8	154	232	130	195	116	174	101	152	84.6	127	66.9	100
	9	143	214	121	181	108	162	94.0	141	78.8	118	62.1	93.2
	10	131	196	111	166	99.4	149	86.8	130	72.8	109	57.2	85.8
	11	119	178	101	152	90.9	136	79.5	119	66.6	100	52.2	78.3
	12	107	160	91.5	137	82.4	124	72.2	108	60.5	90.8	47.3	70.9
	13	95.2	143	82.0	123	74.0	111	65.0	97.5	54.5	81.8	42.4	63.6
	14	84.0	126	72.9	109	65.9	98.9	58.0	87.0	48.7	73.1	37.7	56.6
	15	73.4	110	64.1	96.1	58.2	87.3	51.3	77.0	43.2	64.7	33.2	49.8
	16	64.5	96.8	56.3	84.5	51.1	76.7	45.1	67.7	37.9	56.9	29.2	43.8
	17	57.1	85.7	49.9	74.8	45.3	67.9	40.0	60.0	33.6	50.4	25.8	38.8
	18	51.0	76.5	44.5	66.8	40.4	60.6	35.7	53.5	30.0	44.9	23.0	34.6
	19	45.8	68.6	39.9	59.9	36.3	54.4	32.0	48.0	26.9	40.3	20.7	31.0
	20	41.3	61.9	36.0	54.1	32.7	49.1	28.9	43.3	24.3	36.4	18.7	28.0
	21	37.5	56.2	32.7	49.0	29.7	44.5	26.2	39.3	22.0	33.0	16.9	25.4
	22	34.1	51.2	29.8	44.7	27.0	40.6	23.9	35.8	20.1	30.1	15.4	23.1
	23	31.2	46.8	27.3	40.9	24.7	37.1	21.8	32.8	18.3	27.5	14.1	21.2
	24	28.7	43.0	25.0	37.5	22.7	34.1	20.1	30.1	16.8	25.3	13.0	19.4
	25	26.4	39.6	23.1	34.6	20.9	31.4	18.5	27.7	15.5	23.3	11.9	17.9
	26					19.4	29.0	17.1	25.6	14.4	21.5	11.0	16.6
	27									13.3	20.0	10.2	15.4
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	33.6	50.5	27.3	41.0	23.6	35.4	19.6	29.4	15.2	22.8	10.5	15.7
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	25.2	37.9	20.5	30.8	17.8	26.7	14.8	22.2	11.5	17.3	7.94	11.9
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	1060		927		844		747		631		490	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	543		474		429		379		318		245	
r_{mx}/r_{my}		1.40		1.40		1.40		1.40		1.41		1.41	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

4

COMPOSITE
HSS6

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS6×3×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		24.9		19.7		16.9		13.9		10.7		7.30	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	177	266	147	220	130	195	112	168	92.9	139	73.1	110
	1	176	264	146	218	129	193	111	167	92.2	138	72.6	109
	2	172	257	142	213	126	189	109	163	90.3	135	71.0	107
	3	165	247	137	205	121	182	105	157	87.1	131	68.5	103
	4	155	233	130	194	115	173	99.7	150	82.9	124	65.1	97.6
	5	144	216	121	181	108	162	93.4	140	77.7	117	60.9	91.4
	6	132	198	111	167	99.2	149	86.2	129	71.8	108	56.3	84.4
	7	118	177	101	151	90.0	135	78.4	118	65.5	98.2	51.2	76.8
	8	104	157	89.5	134	80.5	121	70.3	105	58.8	88.2	45.9	68.8
	9	90.8	136	78.5	118	70.9	106	62.1	93.2	52.1	78.2	40.5	60.8
	10	77.6	116	67.8	102	61.5	92.3	54.1	81.2	45.5	68.2	35.3	53.0
	11	65.1	97.6	57.7	86.6	52.6	78.9	46.4	69.7	39.2	58.7	30.3	45.5
	12	54.7	82.0	48.5	72.8	44.3	66.4	39.2	58.9	33.2	49.8	25.6	38.4
	13	46.6	69.9	41.3	62.0	37.7	56.6	33.4	50.2	28.3	42.4	21.8	32.7
	14	40.2	60.3	35.6	53.4	32.5	48.8	28.8	43.3	24.4	36.6	18.8	28.2
	15	35.0	52.5	31.0	46.6	28.4	42.5	25.1	37.7	21.2	31.8	16.4	24.6
	16	30.8	46.1	27.3	40.9	24.9	37.4	22.1	33.1	18.7	28.0	14.4	21.6
	17	27.2	40.9	24.2	36.2	22.1	33.1	19.6	29.3	16.5	24.8	12.8	19.1
	18	24.3	36.4	21.6	32.3	19.7	29.5	17.4	26.2	14.7	22.1	11.4	17.1
	19			19.3	29.0	17.7	26.5	15.7	23.5	13.2	19.8	10.2	15.3
	20							14.1	21.2	11.9	17.9	9.22	13.8
	21											8.36	12.5

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	27.7	41.7	22.7	34.2	19.8	29.7	16.5	24.8	12.8	19.3	8.89	13.4
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	16.7	25.1	13.8	20.8	12.1	18.2	10.1	15.2	7.91	11.9	5.51	8.28
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	824		730		667		593		505		395	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	259		229		209		186		157		121	
r_{mx}/r_{my}		1.78		1.79		1.79		1.79		1.79		1.81	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

Shape		HSS5×4×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		24.9		19.7		16.9		13.9		10.7		7.30	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	181	271	150	225	133	200	115	173	96.3	144	76.5	115
	1	180	270	149	224	133	199	115	172	95.9	144	76.2	114
	2	177	266	147	221	131	196	113	170	94.7	142	75.2	113
	3	173	260	144	216	128	192	111	166	92.7	139	73.5	110
	4	167	251	139	209	124	186	108	162	89.9	135	71.3	107
	5	160	240	134	201	119	179	104	155	86.5	130	68.5	103
	6	152	228	127	191	114	170	98.7	148	82.5	124	65.3	97.9
	7	143	214	120	180	107	161	93.3	140	78.1	117	61.6	92.5
	8	133	199	112	168	100	150	87.4	131	73.2	110	57.7	86.5
	9	122	183	104	156	93.0	140	81.2	122	68.1	102	53.5	80.3
	10	112	167	95.2	143	85.5	128	74.8	112	62.8	94.2	49.2	73.8
	11	101	151	86.5	130	77.9	117	68.3	102	57.4	86.1	44.9	67.3
	12	90.2	135	77.9	117	70.3	106	61.8	92.7	52.0	78.0	40.5	60.8
	13	79.9	120	69.5	104	63.0	94.4	55.4	83.1	46.7	70.1	36.3	54.5
	14	70.1	105	61.5	92.2	55.9	83.8	49.3	74.0	41.6	62.4	32.2	48.4
	15	61.1	91.6	53.8	80.7	49.0	73.5	43.4	65.1	36.7	55.1	28.3	42.5
	16	53.7	80.5	47.3	70.9	43.1	64.6	38.1	57.2	32.3	48.4	24.9	37.3
	17	47.5	71.3	41.9	62.8	38.2	57.2	33.8	50.7	28.6	42.9	22.0	33.1
	18	42.4	63.6	37.3	56.0	34.0	51.0	30.1	45.2	25.5	38.2	19.7	29.5
	19	38.1	57.1	33.5	50.3	30.5	45.8	27.0	40.6	22.9	34.3	17.6	26.5
	20	34.4	51.5	30.3	45.4	27.6	41.3	24.4	36.6	20.7	31.0	15.9	23.9
	21	31.2	46.7	27.4	41.2	25.0	37.5	22.1	33.2	18.7	28.1	14.4	21.7
	22	28.4	42.6	25.0	37.5	22.8	34.2	20.2	30.3	17.1	25.6	13.2	19.7
	23	26.0	39.0	22.9	34.3	20.8	31.3	18.5	27.7	15.6	23.4	12.0	18.1
	24	23.9	35.8	21.0	31.5	19.1	28.7	16.9	25.4	14.3	21.5	11.1	16.6
	25			19.4	29.0	17.6	26.5	15.6	23.4	13.2	19.8	10.2	15.3
	26							14.4	21.7	12.2	18.3	9.42	14.1
	27											8.74	13.1

Properties

M_{nx}/Ω_b	ϕ_bM_{nx}	kip-ft	25.1	37.8	20.6	30.9	17.9	26.9	14.9	22.4	11.6	17.4	8.03	12.1
M_{ny}/Ω_b	ϕ_bM_{ny}	kip-ft	21.5	32.2	17.6	26.5	15.3	23.0	12.8	19.2	10.0	15.0	6.90	10.4
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	652		575		524		465		395		305	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	451		397		361		321		271		209	
r_{mx}/r_{my}			1.20		1.20		1.20		1.20		1.21		1.21	

ASD	LRFD	
$\Omega_c = 2.00$	$\phi_c = 0.75$	Note: Heavy line indicates *Kl/r* equal to or greater than 200.

4

COMPOSITE
HSS5

Table 4-13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS5×3×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		21.5		17.2		14.8		12.2		9.43		6.45	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	152	229	127	190	112	168	97.0	145	80.4	121	63.1	94.6
	1	151	227	126	188	111	167	96.3	144	79.8	120	62.6	93.9
	2	147	221	123	184	109	163	94.1	141	78.1	117	61.2	91.8
	3	141	212	118	177	105	157	90.6	136	75.2	113	59.0	88.5
	4	133	199	111	167	99.2	149	86.0	129	71.5	107	56.0	84.0
	5	123	184	104	156	92.6	139	80.4	121	66.9	100	52.4	78.6
	6	112	168	95.0	142	85.0	128	74.0	111	61.7	92.6	48.3	72.5
	7	99.9	150	85.6	128	76.9	115	67.1	101	56.1	84.2	43.9	65.8
	8	87.8	132	75.9	114	68.5	103	60.0	90	50.3	75.4	39.3	58.9
	9	75.8	114	66.3	99.4	60.0	90.1	52.8	79.2	44.4	66.6	34.6	51.9
	10	64.3	96.5	56.9	85.4	51.8	77.8	45.8	68.7	38.6	57.9	30.1	45.1
	11	53.6	80.4	48.0	72.1	44.1	66.1	39.1	58.7	33.1	49.6	25.7	38.6
	12	45.0	67.6	40.4	60.6	37.0	55.6	32.9	49.4	27.9	41.9	21.7	32.6
	13	38.4	57.6	34.4	51.6	31.6	47.3	28.1	42.1	23.8	35.7	18.5	27.7
	14	33.1	49.6	29.7	44.5	27.2	40.8	24.2	36.3	20.5	30.8	16.0	23.9
	15	28.8	43.2	25.8	38.8	23.7	35.6	21.1	31.6	17.9	26.8	13.9	20.8
	16	25.3	38.0	22.7	34.1	20.8	31.2	18.5	27.8	15.7	23.6	12.2	18.3
	17	22.4	33.7	20.1	30.2	18.5	27.7	16.4	24.6	13.9	20.9	10.8	16.2
	18	20.0	30.0	17.9	26.9	16.5	24.7	14.6	22.0	12.4	18.6	9.63	14.5
	19			16.1	24.2	14.8	22.2	13.1	19.7	11.1	16.7	8.66	13.0
	20									10.1	15.1	7.82	11.7
Properties													
M_{nx}/Ω_b $\phi_b M_{nx}$ kip-ft		20.3	30.5	16.8	25.3	14.7	22.1	12.4	18.6	9.66	14.5	6.73	10.1
M_{ny}/Ω_b $\phi_b M_{ny}$ kip-ft		14.0	21.1	11.7	17.6	10.3	15.4	8.65	13.0	6.79	10.2	4.74	7.13
$P_{ex}(K_xL_x)^2/10^4$ kip-in.^{2}		497		446		409		364		311		243	
$P_{ey}(K_yL_y)^2/10^4$ kip-in.^{2}		213		192		176		156		132		103	
r_{mx}/r_{my}		1.53		1.53		1.53		1.53		1.53		1.54	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

Shape		HSS5×2½×						HSS4×3×					
		1/4		3/16		1/8		3/8		5/16		1/4	
t_{design}, in.		0.233		0.174		0.116		0.349		0.291		0.233	
Steel Wt/ft		11.3		8.79		6.02		14.6		12.7		10.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	87.8	132	72.4	109	56.3	84.5	107	160	94.8	142	82.0	123
	1	86.8	130	71.7	108	55.8	83.7	106	159	94.1	141	81.3	122
	2	84.1	126	69.5	104	54.1	81.1	103	155	91.8	138	79.4	119
	3	79.7	120	66.0	98.9	51.3	77.0	98.9	148	88.1	132	76.4	115
	4	74.0	111	61.3	92.0	47.8	71.7	93.2	140	83.3	125	72.3	108
	5	67.2	101	55.9	83.8	43.5	65.3	86.4	130	77.4	116	67.4	101
	6	59.7	89.6	49.8	74.8	38.9	58.3	78.8	118	70.8	106	61.8	92.7
	7	52.0	78.0	43.5	65.3	34.0	51.0	70.6	106	63.7	95.6	55.8	83.7
	8	44.3	66.4	37.3	55.9	29.1	43.7	62.2	93.3	56.4	84.6	49.6	74.4
	9	36.9	55.4	31.2	46.9	24.4	36.6	53.9	80.8	49.1	73.7	43.4	65.1
	10	30.1	45.2	25.6	38.4	20.0	30.1	45.9	68.8	42.1	63.2	37.4	56.1
	11	24.9	37.3	21.2	31.7	16.6	24.8	38.4	57.5	35.4	53.2	31.7	47.6
	12	20.9	31.4	17.8	26.7	13.9	20.9	32.2	48.4	29.8	44.7	26.6	40.0
	13	17.8	26.7	15.2	22.7	11.9	17.8	27.5	41.2	25.4	38.1	22.7	34.1
	14	15.4	23.1	13.1	19.6	10.2	15.3	23.7	35.5	21.9	32.8	19.6	29.4
	15	13.4	20.1	11.4	17.1	8.91	13.4	20.6	30.9	19.1	28.6	17.1	25.6
	16	11.8	17.6	10.0	15.0	7.83	11.7	18.1	27.2	16.7	25.1	15.0	22.5
	17			8.86	13.3	6.93	10.4	16.1	24.1	14.8	22.3	13.3	19.9
	18							14.3	21.5	13.2	19.9	11.8	17.8
	19											10.6	15.9

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	11.1	16.7	8.70	13.1	6.08	9.14	11.7	17.7	10.4	15.6	8.76	13.2
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	6.78	10.2	5.35	8.04	3.76	5.65	9.58	14.4	8.47	12.7	7.17	10.8
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	315		269		213		245		226		203	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	98.9		84.1		65.8		153		141		126	
r_{mx}/r_{my}			1.79		1.79		1.80		1.27		1.27		1.27	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

4

COMPOSITE HSS4

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS4×3×				HSS4×2$^1/_2$×							
		$^3/_{16}$		$^1/_8$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$	
t_{design}, in.		0.174		0.116		0.349		0.291		0.233		0.174	
Steel Wt/ft		8.15		5.60		13.4		11.6		9.63		7.51	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	67.9	102	53.0	79.5	95.9	144	85.2	128	73.6	110	60.8	91.1
	1	67.4	101	52.6	78.9	94.7	142	84.2	126	72.8	109	60.1	90.2
	2	65.8	98.7	51.4	77.2	91.3	137	81.3	122	70.4	106	58.2	87.3
	3	63.4	95.1	49.5	74.3	85.9	129	76.8	115	66.6	99.8	55.1	82.7
	4	60.1	90.1	47.0	70.4	78.9	118	70.8	106	61.6	92.3	51.1	76.7
	5	56.1	84.1	43.9	65.8	70.7	106	63.7	95.6	55.7	83.5	46.4	69.6
	6	51.6	77.4	40.4	60.5	61.9	92.8	56.1	84.1	49.3	73.9	41.2	61.9
	7	46.7	70.1	36.6	54.8	52.8	79.2	48.2	72.3	42.6	63.9	35.9	53.8
	8	41.7	62.5	32.6	48.9	44.0	66.0	40.5	60.7	36.0	54.1	30.5	45.8
	9	36.6	54.9	28.7	43.0	35.7	53.6	33.2	49.9	29.8	44.7	25.4	38.1
	10	31.7	47.5	24.8	37.2	28.9	43.4	26.9	40.4	24.2	36.3	20.7	31.1
	11	27.0	40.5	21.2	31.8	23.9	35.9	22.2	33.4	20.0	30.0	17.1	25.7
	12	22.7	34.1	17.8	26.7	20.1	30.1	18.7	28.0	16.8	25.2	14.4	21.6
	13	19.3	29.0	15.2	22.8	17.1	25.7	15.9	23.9	14.3	21.5	12.3	18.4
	14	16.7	25.0	13.1	19.6	14.8	22.1	13.7	20.6	12.4	18.5	10.6	15.9
	15	14.5	21.8	11.4	17.1	12.9	19.3	12.0	17.9	10.8	16.1	9.21	13.8
	16	12.8	19.2	10.0	15.0					9.46	14.2	8.09	12.1
	17	11.3	17.0	8.87	13.3								
	18	10.1	15.1	7.92	11.9								
	19	9.06	13.6	7.10	10.7								
	20			6.41	9.62								

Properties

M_{nx}/Ω_b	ϕ_bM_{nx}	kip-ft	6.90	10.4	4.84	7.27	10.3	15.5	9.12	13.7	7.75	11.6	6.13	9.22
M_{ny}/Ω_b	ϕ_bM_{ny}	kip-ft	5.66	8.50	3.97	5.97	7.34	11.0	6.53	9.82	5.57	8.37	4.42	6.65
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	173		137		207		193		174		148	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	107		84.2		95.0		88.4		79.5		68.0	
r_{mx}/r_{my}			1.27		1.27		1.48		1.48		1.48		1.48	

ASD	LRFD
Ω_c = 2.00	ϕ_c = 0.75

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–13 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

4
COMPOSITE
HSS4

Shape		HSS4×2½×		HSS4×2×									
		1/8		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.116		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		5.17		12.1		10.5		8.78		6.87		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	47.2	70.7	85.0	128	75.6	113	65.2	97.9	53.7	80.5	41.3	61.9
	1	46.7	70.0	83.4	125	74.3	111	64.1	96.2	52.8	79.2	40.6	60.9
	2	45.2	67.8	78.7	118	70.3	105	60.9	91.3	50.2	75.3	38.7	58.1
	3	42.9	64.3	71.5	107	64.2	96.2	55.8	83.7	46.2	69.3	35.7	53.6
	4	39.8	59.7	62.4	93.6	56.4	84.7	49.4	74.1	41.2	61.7	31.9	47.9
	5	36.2	54.3	52.5	78.7	47.9	71.8	42.3	63.4	35.5	53.2	27.6	41.4
	6	32.2	48.3	42.4	63.6	39.2	58.8	34.9	52.4	29.6	44.3	23.2	34.7
	7	28.1	42.1	33.0	49.5	30.9	46.3	27.9	41.8	23.8	35.7	18.8	28.2
	8	24.0	35.9	25.2	37.9	23.7	35.6	21.6	32.4	18.6	27.9	14.7	22.1
	9	20.0	30.0	19.9	29.9	18.8	28.1	17.0	25.6	14.7	22.0	11.6	17.5
	10	16.4	24.5	16.2	24.2	15.2	22.8	13.8	20.7	11.9	17.8	9.43	14.2
	11	13.5	20.3	13.4	20.0	12.6	18.8	11.4	17.1	9.83	14.7	7.80	11.7
	12	11.4	17.0	11.2	16.8	10.5	15.8	9.59	14.4	8.26	12.4	6.55	9.83
	13	9.68	14.5							7.03	10.6	5.58	8.37
	14	8.35	12.5										
	15	7.27	10.9										
	16	6.39	9.59										
	17	5.66	8.49										
Properties													
M_{nx}/Ω_b $\phi_b M_{nx}$	kip-ft	4.32	6.49	8.82	13.3	7.88	11.8	6.74	10.1	5.37	8.07	3.80	5.71
M_{ny}/Ω_b $\phi_b M_{ny}$	kip-ft	3.13	4.70	5.30	7.96	4.76	7.16	4.10	6.17	3.29	4.94	2.34	3.52
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	119		168		158		144		124		100	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	53.7		53.0		49.8		45.4		39.0		31.0	
r_{mx}/r_{my}		1.49		1.78		1.78		1.78		1.79		1.79	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

5

COMPOSITE
HSS20–HSS16

Table 4–14
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS20×12×						HSS16×12×					
		5/8		1/2		3/8		5/8		1/2		3/8	
t_{design}, in.		0.581		0.465		0.349		0.581		0.465		0.349	
Steel Wt/ft		127		103		78.4		110		89.6		68.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	1240	1860	1100	1650	959	1440	1040	1560	920	1380	798	1200
	6	1220	1820	1080	1620	941	1410	1020	1530	904	1360	784	1180
	7	1210	1810	1070	1610	935	1400	1010	1520	898	1350	778	1170
	8	1200	1800	1070	1600	928	1390	1010	1510	891	1340	772	1160
	9	1190	1790	1060	1590	920	1380	998	1500	884	1330	765	1150
	10	1180	1770	1050	1570	911	1370	988	1480	875	1310	758	1140
	11	1170	1750	1040	1550	901	1350	978	1470	866	1300	750	1120
	12	1150	1730	1030	1540	891	1340	967	1450	856	1280	741	1110
	13	1140	1710	1010	1520	879	1320	955	1430	846	1270	731	1100
	14	1130	1690	999	1500	867	1300	943	1410	834	1250	721	1080
	15	1110	1670	985	1480	854	1280	929	1390	822	1230	710	1070
	16	1090	1640	970	1460	841	1260	915	1370	810	1210	699	1050
	17	1080	1620	955	1430	827	1240	900	1350	796	1190	687	1030
	18	1060	1590	938	1410	812	1220	885	1330	783	1170	675	1010
	19	1040	1560	922	1380	797	1200	869	1300	768	1150	662	993
	20	1020	1530	904	1360	781	1170	852	1280	753	1130	649	973
	21	1000	1500	886	1330	765	1150	835	1250	738	1110	635	952
	22	981	1470	868	1300	748	1120	818	1230	722	1080	621	931
	23	960	1440	849	1270	731	1100	800	1200	706	1060	607	910
	24	939	1410	829	1240	714	1070	782	1170	690	1030	592	888
	25	917	1380	810	1210	696	1040	763	1140	673	1010	577	866
	26	895	1340	790	1180	678	1020	744	1120	656	984	562	843
	27	872	1310	769	1150	660	990	725	1090	639	959	547	820
	28	850	1270	749	1120	642	963	705	1060	622	933	531	797
	29	827	1240	728	1090	623	935	686	1030	604	906	516	774
	30	804	1210	708	1060	605	907	666	999	587	880	500	751
	32	757	1140	666	999	568	852	627	940	551	827	469	704
	34	711	1070	624	936	531	796	587	881	516	774	438	657
	36	665	997	583	874	494	741	548	822	481	722	407	611
	38	619	929	542	813	458	687	509	764	447	670	377	566
	40	575	862	502	753	423	634	472	707	413	620	348	522
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	528	794	432	649	331	497	379	569	310	466	239	359
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	372	559	303	455	234	352	310	466	255	383	196	295
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	74200		64800		54600		41200		36100		30400	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	31100		27100		22600		25300		22200		18600	
r_{mx}/r_{my}		1.54		1.55		1.55		1.27		1.28		1.28	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 46 ksi
f'_c = 5 ksi

Table 4-14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

5

COMPOSITE
HSS16–HSS14

Shape		HSS16×12×		HSS16×8×								HSS14×10×	
		5/16		5/8		1/2		3/8		5/16		5/8	
t_{design}, in.		0.291		0.581		0.465		0.349		0.291		0.581	
Steel Wt/ft		57.4		93.1		75.9		58.1		48.9		93.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	736	1100	805	1210	707	1060	604	906	551	827	831	1250
	6	722	1080	775	1160	680	1020	581	872	530	795	810	1210
	7	717	1080	765	1150	671	1010	573	860	522	784	803	1200
	8	711	1070	753	1130	661	991	564	846	514	771	794	1190
	9	705	1060	739	1110	649	973	554	831	504	757	785	1180
	10	698	1050	724	1090	636	954	543	814	494	741	774	1160
	11	690	1030	709	1060	622	933	531	796	483	724	763	1140
	12	681	1020	691	1040	607	911	518	776	471	706	750	1130
	13	672	1010	673	1010	591	887	504	756	458	687	737	1110
	14	663	994	654	982	575	862	490	734	445	667	723	1080
	15	653	979	635	952	558	836	475	712	431	646	708	1060
	16	642	963	614	921	540	810	459	689	416	625	693	1040
	17	631	946	593	890	521	782	443	665	402	602	677	1020
	18	619	928	571	857	502	754	427	640	386	580	660	991
	19	607	910	550	824	483	725	410	615	371	557	643	965
	20	594	891	527	791	464	696	393	590	356	533	626	939
	21	581	872	505	757	444	666	376	565	340	510	608	912
	22	568	852	482	724	424	637	359	539	324	486	590	884
	23	555	832	460	690	405	607	342	514	309	463	571	857
	24	541	811	438	656	385	578	326	488	293	440	552	829
	25	527	790	415	623	366	549	309	463	278	417	534	800
	26	513	769	394	590	347	520	293	439	263	394	515	772
	27	498	748	372	558	328	492	276	415	248	372	496	743
	28	484	726	351	527	310	464	261	391	233	350	477	715
	29	469	704	331	496	291	437	245	368	219	329	458	687
	30	455	682	310	465	274	411	230	344	205	308	439	659
	32	426	638	273	409	241	361	202	303	180	270	402	603
	34	397	595	241	362	213	320	179	268	160	239	366	549
	36	368	552	215	323	190	285	159	239	142	214	332	497
	38	340	510	193	290	171	256	143	215	128	192	298	447
	40	313	469	174	262	154	231	129	194	115	173	269	403
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	201	303	296	445	243	366	188	283	159	240	275	414
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	166	249	182	273	150	226	117	175	98.8	148	218	328
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	27200		29500		26200		22300		19900		24900	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	16600		9170		8090		6780		6060		14100	
r_{mx}/r_{my}		1.28		1.79		1.80		1.81		1.81		1.33	

ASD	LRFD
Ω_c = 2.00	ϕ_c = 0.75

COMPOSITE
HSS14–HSS12

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS14×10×								HSS12×10×			
		1/2		3/8		5/16		1/4		1/2		3/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.465		0.349	
Steel Wt/ft		75.9		58.1		48.9		39.5		69.1		52.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	732	1100	630	944	577	865	523	785	651	976	558	837
	6	714	1070	613	920	562	842	509	763	634	951	543	815
	7	707	1060	608	911	556	834	504	755	628	942	538	807
	8	700	1050	601	902	550	825	498	747	621	932	532	798
	9	692	1040	594	890	543	815	491	737	614	921	526	788
	10	682	1020	586	878	535	803	484	726	605	908	518	777
	11	672	1010	577	865	527	791	476	715	596	895	510	766
	12	662	992	567	851	518	777	468	702	587	880	502	753
	13	650	975	557	835	509	763	459	689	576	864	493	739
	14	638	957	546	819	498	748	450	674	565	847	483	724
	15	625	937	535	802	488	732	440	659	553	830	473	709
	16	611	917	523	784	477	715	429	644	541	811	462	693
	17	597	896	510	766	465	698	418	628	528	792	451	676
	18	583	874	498	747	453	680	407	611	515	773	439	659
	19	568	852	485	727	441	661	396	594	501	752	428	641
	20	552	828	471	706	428	642	384	576	488	731	416	623
	21	537	805	457	686	415	623	372	558	473	710	403	605
	22	521	781	443	665	402	603	360	540	459	688	391	586
	23	504	756	429	643	389	583	347	521	444	666	378	567
	24	488	732	415	622	375	563	335	503	429	644	365	548
	25	471	707	400	600	362	543	323	484	414	622	352	528
	26	455	682	386	578	348	523	310	465	399	599	339	509
	27	438	657	371	556	335	502	298	447	384	577	326	489
	28	421	632	356	535	322	482	285	428	369	554	313	470
	29	405	607	342	513	308	462	273	409	355	532	300	450
	30	388	582	328	492	295	442	261	391	340	510	288	431
	32	356	534	299	449	269	403	237	355	311	466	262	394
	34	324	486	272	408	244	365	214	321	282	424	238	357
	36	294	441	246	369	219	329	192	287	255	383	215	322
	38	264	396	221	331	197	295	172	258	229	344	193	289
	40	238	358	199	299	177	266	155	233	207	310	174	261
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	227	341	175	263	148	223	120	181	181	272	140	211
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	180	271	139	209	118	177	95.8	144	160	240	124	186
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	22200		18600		16600		14500		14800		12500	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	12500		10500		9330		8150		10900		9130	
r_{mx}/r_{my}		1.33		1.33		1.33		1.34		1.17		1.17	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 46 ksi
f'_c = 5 ksi

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

5

COMPOSITE
HSS12

Shape		HSS12×10×				HSS12×8×							
		5/16		1/4		5/8		1/2		3/8		1/4	
t_{design}, in.		0.291		0.233		0.581		0.465		0.349		0.233	
Steel Wt/ft		44.6		36.0		76.1		62.3		47.8		32.6	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	510	765	461	692	640	961	561	842	478	717	391	586
	6	496	745	448	673	615	923	539	809	459	689	375	562
	7	492	737	444	666	606	910	532	798	453	679	369	554
	8	486	729	439	658	597	895	523	785	445	668	363	544
	9	480	720	433	649	585	878	514	770	437	655	356	534
	10	473	710	427	640	573	860	503	754	428	642	348	522
	11	466	698	420	630	560	840	492	737	418	627	340	509
	12	458	686	412	618	546	819	479	719	408	611	331	496
	13	449	674	404	606	531	796	466	700	397	595	321	482
	14	440	660	396	594	515	773	453	679	385	577	311	467
	15	431	646	387	581	499	749	439	658	373	559	301	452
	16	421	631	378	567	482	723	424	636	360	540	290	436
	17	410	615	368	552	465	697	409	614	347	521	280	419
	18	400	599	358	537	447	671	394	591	334	501	268	403
	19	389	583	348	522	429	644	378	567	321	481	257	386
	20	377	566	338	507	411	617	362	544	307	461	246	369
	21	366	549	327	491	393	589	347	520	294	441	234	352
	22	354	531	316	474	375	562	331	496	280	420	223	335
	23	342	514	305	458	356	534	315	472	267	400	212	318
	24	331	496	294	442	338	507	299	449	253	380	200	301
	25	319	478	283	425	320	481	284	425	240	360	189	284
	26	307	460	272	409	303	454	268	402	227	340	179	268
	27	295	442	261	392	285	428	253	380	214	321	168	252
	28	283	424	250	376	269	403	238	357	201	302	158	236
	29	271	406	240	359	252	378	224	336	189	284	147	221
	30	259	389	229	343	236	354	210	314	177	265	138	206
	32	236	354	208	312	207	311	184	276	155	233	121	181
	34	214	320	187	281	184	275	163	245	138	206	107	161
	36	192	288	168	252	164	246	146	218	123	184	95.5	143
	38	172	258	150	226	147	220	131	196	110	165	85.7	129
	40	155	233	136	204	133	199	118	177	99.4	149	77.4	116

Properties

M_{nx}/Ω_b	ϕ_bM_{nx} kip-ft	119	179	96.6	145	188	283	156	235	122	183	84.0	126
M_{ny}/Ω_b	ϕ_bM_{ny} kip-ft	105	158	85.4	128	142	214	118	178	92.1	138	63.8	95.8
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	11100		9760		13800		12200		10400		8160	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	8160		7140		6970		6190		5230		4060	
r_{mx}/r_{my}		1.17		1.17		1.40		1.41		1.41		1.42	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

5

COMPOSITE
HSS12–HSS10

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS12×6×								HSS10×8×			
		5/8		1/2		3/8		1/4		5/8		1/2	
t_{design}, in.		0.581		0.465		0.349		0.233		0.581		0.465	
Steel Wt/ft		67.6		55.5		42.7		29.2		67.6		55.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	541	811	471	707	398	596	320	480	558	837	488	732
	6	505	757	440	660	372	558	299	448	535	803	469	703
	7	492	738	430	644	363	544	291	437	527	791	462	693
	8	478	717	418	626	353	529	283	424	518	778	454	681
	9	463	694	404	607	342	513	274	411	508	763	446	668
	10	446	670	390	585	330	495	264	396	497	746	436	654
	11	429	643	375	562	317	476	253	380	486	728	426	639
	12	410	615	359	539	304	456	242	364	473	709	415	622
	13	391	586	342	514	290	435	231	347	460	689	403	605
	14	371	557	325	488	276	413	219	329	446	668	391	587
	15	351	527	308	462	261	392	207	311	431	646	379	568
	16	331	496	291	436	246	370	195	293	416	624	366	549
	17	311	466	273	410	232	348	183	275	401	601	352	529
	18	290	436	256	383	217	325	171	257	385	577	339	508
	19	270	406	238	358	202	304	159	239	369	553	325	487
	20	251	376	221	332	188	282	148	222	353	529	311	466
	21	232	348	205	307	174	262	137	205	336	505	297	445
	22	214	320	189	283	161	241	126	188	320	480	283	424
	23	195	293	173	260	148	221	115	172	304	456	269	403
	24	180	269	159	239	136	203	106	158	288	432	255	382
	25	165	248	147	220	125	187	97.3	146	272	409	241	362
	26	153	229	136	203	116	173	90.0	135	257	386	228	342
	27	142	213	126	188	107	161	83.4	125	242	363	214	322
	28	132	198	117	175	100	149	77.6	116	227	341	202	302
	29	123	184	109	163	92.8	139	72.3	108	212	319	189	283
	30	115	172	102	153	86.8	130	67.6	101	198	298	176	265
	32	101	151	89.5	134	76.2	114	59.4	89.1	174	262	155	233
	34	89.5	134	79.2	119	67.5	101	52.6	78.9	155	232	137	206
	36	79.8	120	70.7	106	60.2	90.4	46.9	70.4	138	207	123	184
	38	71.6	107	63.4	95.2	54.1	81.1	42.1	63.2	124	186	110	165
	40			57.3	85.9	48.8	73.2	38.0	57.0	112	167	99.3	149

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	158	237	132	198	103	155	71.4	107	143	215	119	179
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	96.6	145	80.9	122	63.5	95.4	44.3	66.5	122	184	102	153
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	10800		9660		8310		6550		8520		7590	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	3400		3010		2560		2000		5860		5210	
r_{mx}/r_{my}		1.79		1.79		1.80		1.81		1.21		1.21	

ASD	LRFD	
$\Omega_c = 2.00$	$\phi_c = 0.75$	Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

5

COMPOSITE
HSS10

Shape		HSS10×8×								HSS10×6×			
		3/8		5/16		1/4		3/16		5/8		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.581		0.465	
Steel Wt/ft		42.7		36.1		29.2		22.2		59.1		48.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	415	622	376	565	337	506	296	445	467	700	407	610
	6	398	597	361	542	323	485	284	425	435	652	379	569
	7	392	589	356	534	318	478	279	419	423	635	370	555
	8	386	579	350	525	313	469	274	411	411	616	359	539
	9	379	568	343	515	307	460	268	402	397	596	348	521
	10	371	556	336	504	300	450	262	393	383	574	335	503
	11	362	543	328	492	293	439	255	383	367	550	322	482
	12	353	529	319	479	285	427	248	372	351	526	308	461
	13	343	515	310	466	277	415	241	361	334	500	293	439
	14	333	499	301	452	268	402	233	349	316	474	278	417
	15	322	483	291	437	259	389	225	337	298	448	263	394
	16	311	467	281	422	250	375	216	324	280	421	247	371
	17	300	450	271	406	240	360	208	311	263	394	232	348
	18	288	432	260	390	231	346	199	298	245	368	217	325
	19	277	415	249	374	221	331	190	285	228	341	202	303
	20	265	397	239	358	211	316	181	272	211	316	187	281
	21	253	379	228	342	201	302	172	258	194	291	173	259
	22	241	361	217	325	191	287	163	245	178	267	159	238
	23	229	344	206	309	181	272	155	232	163	244	145	218
	24	217	326	195	293	172	257	146	219	150	224	133	200
	25	206	309	185	277	162	243	137	206	138	207	123	184
	26	194	291	174	261	153	229	129	193	127	191	114	171
	27	183	275	164	246	143	215	121	181	118	177	105	158
	28	172	258	154	231	134	202	113	169	110	165	98.1	147
	29	161	242	144	216	125	188	105	158	102	154	91.4	137
	30	151	226	135	202	117	176	98.2	147	95.7	144	85.4	128
	32	133	199	118	178	103	155	86.3	129	84.1	126	75.1	113
	34	117	176	105	157	91.3	137	76.4	115	74.5	112	66.5	100
	36	105	157	93.6	140	81.4	122	68.2	102	66.5	100	59.3	89.0
	38	94.0	141	84.0	126	73.1	110	61.2	91.8	59.6	89.5	53.2	79.9
	40	84.8	127	75.8	114	65.9	98.9	55.2	82.8				
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	93.0	140	79.0	119	64.5	97.0	49.1	73.8	118	177	98.7	148
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	79.8	120	67.9	102	55.4	83.3	42.2	63.5	82.1	123	69.1	104
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	6490		5820		5070		4260		6640		5930	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	4450		3990		3470		2890		2820		2520	
r_{mx}/r_{my}		1.21		1.21		1.21		1.21		1.53		1.53	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

5

COMPOSITE HSS10

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS10×6×								HSS10×5×			
		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^3/_8$		$^5/_{16}$	
t_{design}, in.		0.349		0.291		0.233		0.174		0.349		0.291	
Steel Wt/ft		37.6		31.8		25.8		19.7		35.1		29.7	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	343	515	310	464	275	413	239	359	307	461	276	414
	6	320	480	289	434	257	385	222	334	279	418	251	376
	7	312	468	282	423	250	375	217	325	269	404	242	363
	8	303	455	274	411	243	364	210	315	258	388	233	349
	9	294	441	265	398	235	352	203	304	247	370	222	333
	10	283	425	256	383	226	339	195	293	234	352	211	317
	11	272	408	246	368	217	326	187	280	221	332	199	299
	12	260	390	235	352	208	311	178	267	208	312	187	281
	13	248	372	224	336	198	296	169	254	195	292	175	263
	14	236	353	213	319	187	281	160	240	181	271	163	244
	15	223	334	201	302	177	266	151	226	167	251	151	226
	16	210	315	190	284	167	250	142	213	154	231	139	208
	17	197	296	178	267	156	234	132	199	141	211	127	190
	18	184	277	166	250	146	219	123	185	128	192	115	173
	19	172	258	155	233	136	203	114	171	116	173	104	157
	20	159	239	144	216	126	188	106	158	104	156	94.2	141
	21	147	221	133	199	116	174	97.0	146	94.6	142	85.4	128
	22	136	204	122	184	106	160	88.7	133	86.2	129	77.8	117
	23	124	186	112	168	97.4	146	81.1	122	78.8	118	71.2	107
	24	114	171	103	154	89.5	134	74.5	112	72.4	109	65.4	98.1
	25	105	158	94.9	142	82.5	124	68.7	103	66.7	100	60.3	90.4
	26	97.2	146	87.7	132	76.2	114	63.5	95.2	61.7	92.5	55.7	83.6
	27	90.2	135	81.3	122	70.7	106	58.9	88.3	57.2	85.8	51.7	77.5
	28	83.8	126	75.6	113	65.7	98.6	54.7	82.1	53.2	79.8	48.0	72.1
	29	78.2	117	70.5	106	61.3	91.9	51.0	76.5	49.6	74.4	44.8	67.2
	30	73.0	110	65.9	98.8	57.3	85.9	47.7	71.5	46.3	69.5	41.9	62.8
	32	64.2	96.3	57.9	86.9	50.3	75.5	41.9	62.9	40.7	61.1	36.8	55.2
	34	56.9	85.3	51.3	76.9	44.6	66.9	37.1	55.7	36.1	54.1	32.6	48.9
	36	50.7	76.1	45.8	68.6	39.8	59.7	33.1	49.7				
	38	45.5	68.3	41.1	61.6	35.7	53.5	29.7	44.6				
	40	41.1	61.6	37.1	55.6	32.2	48.3	26.8	40.2				

Properties

M_{nx}/Ω_b	ϕ_bM_{nx} kip-ft	77.5	116	66.1	99.3	54.1	81.3	41.3	62.0	69.8	105	59.6	89.5
M_{ny}/Ω_b	ϕ_bM_{ny} kip-ft	54.4	81.8	46.5	69.8	38.1	57.2	29.1	43.8	42.9	64.5	36.7	55.2
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	5130		4650		4050		3400		4410		4020	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	2160		1940		1700		1410		1370		1240	
r_{mx}/r_{my}			1.54		1.55		1.54		1.55		1.79		1.80

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

5

COMPOSITE
HSS10–HSS9

Shape	HSS10×5×				HSS9×7×							
	1/4		3/16		5/8		1/2		3/8		5/16	
t_{design}, in.	0.233		0.174		0.581		0.465		0.349		0.291	
Steel Wt/ft	24.1		18.4		59.1		48.7		37.6		31.8	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft) 0	244	366	211	316	473	710	413	620	349	524	316	474
6	222	332	190	286	448	672	392	588	332	497	300	450
7	214	321	184	275	439	659	384	576	325	488	294	441
8	205	308	176	264	429	644	376	564	318	477	288	432
9	196	294	168	251	419	628	366	550	310	466	281	421
10	186	279	159	238	407	610	356	534	302	453	273	409
11	176	264	150	225	394	591	345	518	293	439	265	397
12	165	248	140	210	380	571	334	501	283	425	256	384
13	154	231	131	196	366	550	322	482	273	410	247	370
14	143	215	121	182	352	528	309	464	263	394	237	356
15	132	199	112	167	337	505	296	444	252	377	227	341
16	122	183	102	153	321	482	283	424	241	361	217	326
17	111	167	93.1	140	305	458	269	404	229	344	207	310
18	101	152	84.3	126	290	435	256	383	218	327	197	295
19	91.2	137	75.8	114	274	411	242	363	206	310	186	279
20	82.3	124	68.4	103	258	387	228	343	195	292	176	264
21	74.7	112	62.0	93.0	243	364	215	322	184	275	166	248
22	68.1	102	56.5	84.8	227	341	202	302	172	259	155	233
23	62.3	93.4	51.7	77.6	212	319	189	283	161	242	146	218
24	57.2	85.8	47.5	71.2	198	297	176	264	151	226	136	204
25	52.7	79.1	43.8	65.7	184	275	164	245	140	211	126	190
26	48.7	73.1	40.5	60.7	170	255	151	227	130	195	117	176
27	45.2	67.8	37.5	56.3	157	236	140	211	121	181	109	163
28	42.0	63.0	34.9	52.3	146	219	131	196	112	168	101	151
29	39.2	58.8	32.5	48.8	136	205	122	183	105	157	94.1	141
30	36.6	54.9	30.4	45.6	127	191	114	171	97.7	147	87.9	132
32	32.2	48.3	26.7	40.1	112	168	100	150	85.9	129	77.3	116
34	28.5	42.7	23.7	35.5	99.2	149	88.5	133	76.0	114	68.5	103
36					88.5	133	79.0	118	67.8	102	61.1	91.6
38					79.4	119	70.9	106	60.9	91.3	54.8	82.2
40					71.7	108	64.0	95.9	54.9	82.4	49.5	74.2
Properties												
M_{nx}/Ω_b $\phi_b M_{nx}$ kip-ft	48.8	73.4	37.3	56.1	111	167	92.9	140	72.9	110	62.2	93.4
M_{ny}/Ω_b $\phi_b M_{ny}$ kip-ft	30.2	45.4	23.2	34.8	93.0	140	78.1	117	61.4	92.3	52.4	78.7
$P_{ex}(K_xL_x)^2/10^4$ kip-in.^{2}	3530		2970		5730		5140		4420		3970	
$P_{ey}(K_yL_y)^2/10^4$ kip-in.^{2}	1080		899		3770		3360		2880		2590	
r_{mx}/r_{my}	1.81		1.82		1.23		1.24		1.24		1.24	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

5

COMPOSITE HSS9

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS9×5×											
		5/8		1/2		3/8		5/16		1/4		3/16	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		50.6		41.9		32.5		27.6		22.4		17.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL)$_y$ with respect to weak axis (ft)	0	386	580	336	504	282	423	253	380	224	336	193	289
	6	348	522	304	456	255	383	230	345	203	304	174	261
	7	335	503	293	439	247	370	222	333	196	294	168	252
	8	321	481	281	421	237	355	213	319	188	282	161	241
	9	305	458	268	402	226	339	203	305	179	269	153	230
	10	289	433	254	381	214	321	193	289	170	255	145	218
	11	272	408	239	359	202	303	182	273	161	241	137	205
	12	254	381	224	336	190	285	171	257	151	226	128	192
	13	236	355	209	314	177	266	160	240	141	211	119	179
	14	219	328	194	291	165	247	148	223	131	196	110	166
	15	201	301	179	268	152	228	137	206	121	181	102	152
	16	184	275	164	246	140	209	126	189	111	166	93.0	140
	17	167	250	149	224	128	191	115	173	101	152	84.7	127
	18	151	226	135	203	116	174	105	157	91.9	138	76.7	115
	19	135	203	122	182	104	157	94.4	142	82.9	124	68.9	103
	20	122	183	110	165	94.3	141	85.2	128	74.8	112	62.1	93.2
	21	111	166	99.5	149	85.5	128	77.3	116	67.8	102	56.4	84.6
	22	101	151	90.7	136	77.9	117	70.4	106	61.8	92.7	51.4	77.0
	23	92.3	138	83.0	124	71.3	107	64.4	96.7	56.5	84.8	47.0	70.5
	24	84.7	127	76.2	114	65.5	98.2	59.2	88.8	51.9	77.9	43.2	64.7
	25	78.1	117	70.2	105	60.3	90.5	54.5	81.8	47.9	71.8	39.8	59.7
	26	72.2	108	64.9	97.4	55.8	83.7	50.4	75.6	44.2	66.4	36.8	55.2
	27	67.0	100	60.2	90.3	51.7	77.6	46.8	70.1	41.0	61.5	34.1	51.1
	28	62.3	93.4	56.0	84.0	48.1	72.1	43.5	65.2	38.1	57.2	31.7	47.6
	29	58.0	87.1	52.2	78.3	44.8	67.3	40.5	60.8	35.6	53.3	29.6	44.3
	30	54.2	81.4	48.8	73.2	41.9	62.9	37.9	56.8	33.2	49.8	27.6	41.4
	32	47.7	71.5	42.9	64.3	36.8	55.2	33.3	49.9	29.2	43.8	24.3	36.4
	34							29.5	44.2	25.9	38.8	21.5	32.3

Properties

			5/8		1/2		3/8		5/16		1/4		3/16	
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	88.3	133	74.7	112	59.1	88.8	50.5	75.9	41.5	62.4	31.8	47.8
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	58.1	87.3	49.3	74.1	39.2	58.9	33.6	50.5	27.7	41.6	21.2	31.9
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	4280		3860		3330		3020		2680		2230	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	1600		1440		1240		1120		982		816	
r_{mx}/r_{my}			1.64		1.64		1.64		1.64		1.65		1.65	

ASD	LRFD
Ω_c = 2.00	ϕ_c = 0.75

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

5

COMPOSITE HSS8

Shape		HSS8×6×											
		5/8		1/2		3/8		5/16		1/4		3/16	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		50.6		41.9		32.5		27.6		22.4		17.1	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	393	589	343	514	288	433	260	390	230	346	199	299
	6	364	547	318	478	269	403	242	363	214	322	185	277
	7	355	532	310	465	262	393	236	354	209	313	180	270
	8	344	516	301	451	254	381	229	344	203	304	175	262
	9	332	498	291	436	246	368	221	332	196	294	168	253
	10	319	478	280	420	237	355	213	320	189	283	162	243
	11	305	458	268	402	227	340	205	307	181	271	155	233
	12	291	436	256	384	217	325	196	293	173	259	148	222
	13	276	414	243	365	206	309	186	279	164	246	140	210
	14	261	392	230	346	196	293	176	265	156	233	133	199
	15	246	369	217	326	185	277	167	250	147	220	125	187
	16	230	346	204	306	174	260	157	235	138	207	117	176
	17	215	323	191	286	163	244	147	220	129	194	109	164
	18	200	300	178	267	152	228	137	206	120	181	102	153
	19	185	278	165	247	141	211	127	191	112	168	94.2	141
	20	171	256	152	229	130	196	118	177	103	155	86.8	130
	21	157	235	140	210	120	180	109	163	95.3	143	79.7	120
	22	143	214	128	193	110	165	99.8	150	87.3	131	72.8	109
	23	131	196	117	176	101	151	91.3	137	79.9	120	66.6	99.9
	24	120	180	108	162	92.7	139	83.9	126	73.4	110	61.1	91.7
	25	111	166	99.4	149	85.4	128	77.3	116	67.6	101	56.4	84.5
	26	102	154	91.9	138	79.0	118	71.5	107	62.5	93.8	52.1	78.1
	27	94.9	142	85.2	128	73.2	110	66.3	99.4	58.0	87.0	48.3	72.5
	28	88.3	132	79.3	119	68.1	102	61.6	92.5	53.9	80.9	44.9	67.4
	29	82.3	123	73.9	111	63.5	95.2	57.5	86.2	50.3	75.4	41.9	62.8
	30	76.9	115	69.0	104	59.3	89.0	53.7	80.5	47.0	70.4	39.1	58.7
	32	67.6	101	60.7	91.0	52.1	78.2	47.2	70.8	41.3	61.9	34.4	51.6
	34	59.9	89.8	53.7	80.6	46.2	69.3	41.8	62.7	36.6	54.8	30.5	45.7
	36	53.4	80.1	47.9	71.9	41.2	61.8	37.3	55.9	32.6	48.9	27.2	40.8
	38			43.0	64.5	37.0	55.5	33.5	50.2	29.3	43.9	24.4	36.6
	40							30.2	45.3	26.4	39.6	22.0	33.0
Properties													
M_{nx}/Ω_b ϕ_bM_{nx}	kip-ft	82.8	124	69.9	105	55.3	83.1	47.3	71.1	38.8	58.4	29.7	44.7
M_{ny}/Ω_b ϕ_bM_{ny}	kip-ft	67.7	102	57.3	86.1	45.4	68.2	38.8	58.4	31.9	48.0	24.5	36.8
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	3660		3300		2840		2580		2260		1890	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	2280		2040		1760		1580		1390		1160	
r_{mx}/r_{my}		1.27		1.27		1.27		1.28		1.28		1.28	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

5

COMPOSITE
HSS8

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS8×4×											
		5/8		1/2		3/8		5/16		1/4		3/16	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		42.1		35.1		27.4		23.3		19.0		14.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	310	465	270	405	225	338	202	302	177	265	151	226
	6	262	393	230	345	193	290	173	260	152	228	129	194
	7	247	370	217	325	183	274	164	246	144	216	122	183
	8	230	345	203	304	171	257	154	231	135	203	115	172
	9	213	319	188	282	159	239	143	215	126	189	107	160
	10	195	292	173	259	147	220	132	198	116	174	98.2	147
	11	176	265	157	236	134	201	121	181	106	159	89.8	135
	12	159	238	142	213	121	182	110	164	96.5	145	81.3	122
	13	141	212	127	191	109	164	98.6	148	86.9	130	73.1	110
	14	124	187	113	169	97.2	146	87.9	132	77.6	116	65.1	97.6
	15	109	163	98.8	148	85.7	129	77.7	117	68.6	103	57.4	86.1
	16	95.4	143	86.9	130	75.3	113	68.3	102	60.3	90.4	50.4	75.7
	17	84.5	127	77.0	115	66.7	100	60.5	90.7	53.4	80.1	44.7	67.0
	18	75.4	113	68.6	103	59.5	89.3	53.9	80.9	47.6	71.5	39.9	59.8
	19	67.6	101	61.6	92.4	53.4	80.1	48.4	72.6	42.8	64.1	35.8	53.7
	20	61.0	91.6	55.6	83.4	48.2	72.3	43.7	65.5	38.6	57.9	32.3	48.4
	21	55.4	83.1	50.4	75.6	43.7	65.6	39.6	59.5	35.0	52.5	29.3	43.9
	22	50.4	75.7	45.9	68.9	39.8	59.8	36.1	54.2	31.9	47.8	26.7	40.0
	23	46.2	69.2	42.0	63.1	36.5	54.7	33.0	49.6	29.2	43.8	24.4	36.6
	24	42.4	63.6	38.6	57.9	33.5	50.2	30.3	45.5	26.8	40.2	22.4	33.6
	25	39.1	58.6	35.6	53.4	30.9	46.3	28.0	41.9	24.7	37.0	20.7	31.0
	26			32.9	49.3	28.5	42.8	25.9	38.8	22.8	34.3	19.1	28.7
	27							24.0	36.0	21.2	31.8	17.7	26.6
	28											16.5	24.7

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	63.0	94.7	53.8	80.9	43.0	64.7	37.0	55.6	30.5	45.9	23.5	35.3
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	38.1	57.2	32.8	49.3	26.4	39.6	22.7	34.2	18.8	28.3	14.5	21.8
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	2560		2330		2030		1840		1640		1390	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	802		730		634		575		507		423	
r_{mx}/r_{my}			1.79		1.79		1.79		1.79		1.80		1.81	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

COMPOSITE HSS7

Shape		HSS7×5×											
		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		35.1		27.4		23.3		19.0		14.5		9.85	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	276	414	232	347	208	312	183	275	157	236	131	196
	6	248	372	209	313	188	282	166	248	142	212	117	175
	7	239	358	201	302	181	271	160	239	136	204	112	168
	8	229	343	193	289	173	260	153	229	131	196	107	161
	9	217	326	184	276	165	248	146	219	124	186	102	152
	10	206	308	174	261	157	235	138	207	118	176	95.8	144
	11	193	290	164	246	148	221	130	195	111	166	89.7	135
	12	181	271	153	230	138	207	122	183	103	155	83.6	125
	13	168	252	143	214	129	193	114	171	96.2	144	77.3	116
	14	155	232	132	198	119	179	105	158	88.9	133	71.1	107
	15	142	213	122	183	110	165	97.1	146	81.8	123	65.0	97.5
	16	130	195	111	167	101	151	89.0	133	74.7	112	59.1	88.6
	17	118	177	101	152	91.7	138	81.1	122	67.9	102	53.3	80.0
	18	106	159	91.7	137	83.0	125	73.5	110	61.3	92.0	47.7	71.6
	19	95.1	143	82.3	123	74.6	112	66.0	99.1	55.0	82.5	42.8	64.3
	20	85.9	129	74.3	111	67.4	101	59.6	89.4	49.7	74.5	38.7	58.0
	21	77.9	117	67.4	101	61.1	91.6	54.1	81.1	45.0	67.6	35.1	52.6
	22	71.0	106	61.4	92.1	55.7	83.5	49.3	73.9	41.0	61.6	32.0	47.9
	23	64.9	97.4	56.2	84.3	50.9	76.4	45.1	67.6	37.5	56.3	29.2	43.9
	24	59.6	89.4	51.6	77.4	46.8	70.2	41.4	62.1	34.5	51.7	26.9	40.3
	25	55.0	82.4	47.5	71.3	43.1	64.7	38.1	57.2	31.8	47.7	24.7	37.1
	26	50.8	76.2	44.0	65.9	39.9	59.8	35.3	52.9	29.4	44.1	22.9	34.3
	27	47.1	70.7	40.8	61.1	37.0	55.4	32.7	49.1	27.2	40.9	21.2	31.8
	28	43.8	65.7	37.9	56.9	34.4	51.5	30.4	45.6	25.3	38.0	19.7	29.6
	29	40.8	61.3	35.3	53.0	32.0	48.1	28.3	42.5	23.6	35.4	18.4	27.6
	30	38.2	57.2	33.0	49.5	29.9	44.9	26.5	39.7	22.1	33.1	17.2	25.8
	32			29.0	43.5	26.3	39.5	23.3	34.9	19.4	29.1	15.1	22.7
	34									17.2	25.8	13.4	20.1

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	50.2	75.4	40.1	60.2	34.4	51.8	28.4	42.7	21.8	32.8	15.0	22.5
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	39.6	59.6	31.7	47.7	27.3	41.1	22.6	33.9	17.4	26.1	11.9	17.9
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	1970		1710		1560		1380		1160		909	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	1130		976		884		783		652		508	
r_{mx}/r_{my}			1.32		1.32		1.33		1.33		1.33		1.34	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

5

COMPOSITE HSS7

Table 4–14 (continued)

Available Strength in Axial Compression, kips

Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS7×4×1/2		HSS7×4×3/8		HSS7×4×5/16		HSS7×4×1/4		HSS7×4×3/16		HSS7×4×1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		31.7		24.9		21.2		17.3		13.3		9.00	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	242	363	202	303	181	271	159	238	135	203	111	166
	6	205	308	173	259	155	232	136	204	115	173	93.9	141
	7	193	290	163	245	146	220	129	193	109	164	88.5	133
	8	181	271	153	229	137	206	121	181	102	153	82.6	124
	9	167	251	142	213	128	191	112	168	95.0	142	76.4	115
	10	153	230	130	196	117	176	103	155	87.4	131	70.0	105
	11	139	209	119	178	107	161	94.5	142	79.8	120	63.6	95.3
	12	125	188	108	161	97.1	146	85.7	128	72.2	108	57.2	85.8
	13	112	168	96.4	145	87.2	131	77.0	115	64.8	97.2	51.0	76.5
	14	98.8	148	85.6	128	77.6	116	68.6	103	57.6	86.4	45.0	67.6
	15	86.4	130	75.3	113	68.4	103	60.5	90.7	50.7	76.0	39.4	59.0
	16	75.9	114	66.1	99.2	60.1	90.1	53.1	79.7	44.5	66.8	34.6	51.9
	17	67.3	101	58.6	87.9	53.2	79.8	47.1	70.6	39.5	59.2	30.6	46.0
	18	60.0	90.0	52.3	78.4	47.5	71.2	42.0	63.0	35.2	52.8	27.3	41.0
	19	53.8	80.8	46.9	70.4	42.6	63.9	37.7	56.5	31.6	47.4	24.5	36.8
	20	48.6	72.9	42.3	63.5	38.4	57.7	34.0	51.0	28.5	42.8	22.1	33.2
	21	44.1	66.1	38.4	57.6	34.9	52.3	30.8	46.3	25.9	38.8	20.1	30.1
	22	40.2	60.2	35.0	52.5	31.8	47.7	28.1	42.2	23.6	35.3	18.3	27.4
	23	36.7	55.1	32.0	48.0	29.1	43.6	25.7	38.6	21.6	32.3	16.7	25.1
	24	33.7	50.6	29.4	44.1	26.7	40.1	23.6	35.4	19.8	29.7	15.4	23.1
	25	31.1	46.7	27.1	40.6	24.6	36.9	21.8	32.7	18.2	27.4	14.2	21.3
	26			25.1	37.6	22.8	34.1	20.1	30.2	16.9	25.3	13.1	19.6
	27							18.7	28.0	15.6	23.5	12.1	18.2
	28											11.3	16.9

Properties

			1/2		3/8		5/16		1/4		3/16		1/8	
M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	43.2	64.9	34.7	52.2	30.0	45.0	24.8	37.3	19.1	28.7	13.2	19.8
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	29.0	43.6	23.4	35.2	20.3	30.5	16.8	25.3	13.0	19.6	8.97	13.5
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	1630		1420		1290		1150		975		763	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	638		556		505		446		374		291	
r_{mx}/r_{my}			1.60		1.60		1.60		1.61		1.61		1.62	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

Shape		HSS6×5×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		31.7		24.9		21.2		17.3		13.3		9.00	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	246	369	206	310	185	278	163	244	139	209	115	173
	1	245	368	206	309	185	277	162	244	139	208	115	172
	2	243	365	204	306	183	275	161	242	138	206	114	170
	3	239	359	201	302	180	271	159	238	136	203	112	168
	4	234	351	197	295	177	265	156	233	133	199	109	164
	5	228	342	192	288	172	258	152	227	129	194	106	160
	6	220	331	186	278	167	250	147	220	125	188	103	154
	7	212	318	179	268	161	241	141	212	121	181	98.7	148
	8	202	304	171	256	154	231	135	203	115	173	94.2	141
	9	192	288	163	244	146	219	129	193	110	165	89.4	134
	10	181	272	154	231	138	208	122	183	104	156	84.2	126
	11	170	255	145	217	130	195	115	172	97.5	146	78.9	118
	12	158	238	135	203	122	183	107	161	91.1	137	73.4	110
	13	147	220	125	188	113	170	100	150	84.7	127	67.9	102
	14	135	203	116	174	105	157	92.5	139	78.2	117	62.4	93.6
	15	124	186	106	160	96.2	144	85.1	128	71.8	108	57.0	85.5
	16	113	169	97.1	146	87.9	132	77.8	117	65.5	98.3	51.7	77.6
	17	102	153	88.1	132	79.9	120	70.7	106	59.5	89.2	46.7	70.0
	18	91.3	137	79.4	119	72.1	108	63.9	95.9	53.6	80.4	41.7	62.6
	19	81.9	123	71.2	107	64.7	97.1	57.4	86.1	48.1	72.1	37.5	56.2
	20	73.9	111	64.3	96.4	58.4	87.6	51.8	77.7	43.4	65.1	33.8	50.7
	21	67.1	101	58.3	87.5	53.0	79.5	47.0	70.4	39.4	59.1	30.7	46.0
	22	61.1	91.7	53.1	79.7	48.3	72.4	42.8	64.2	35.9	53.8	27.9	41.9
	23	55.9	83.9	48.6	72.9	44.2	66.3	39.2	58.7	32.8	49.2	25.6	38.4
	24	51.3	77.0	44.7	67.0	40.6	60.9	36.0	53.9	30.1	45.2	23.5	35.2
	25	47.3	71.0	41.2	61.7	37.4	56.1	33.1	49.7	27.8	41.7	21.6	32.5
	26	43.7	65.6	38.0	57.1	34.6	51.9	30.6	46.0	25.7	38.5	20.0	30.0
	27	40.6	60.9	35.3	52.9	32.1	48.1	28.4	42.6	23.8	35.7	18.6	27.8
	28	37.7	56.6	32.8	49.2	29.8	44.7	26.4	39.6	22.1	33.2	17.3	25.9
	29	35.2	52.7	30.6	45.9	27.8	41.7	24.6	36.9	20.6	31.0	16.1	24.1
	30	32.9	49.3	28.6	42.9	26.0	38.9	23.0	34.5	19.3	28.9	15.0	22.5
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	39.5	59.4	31.8	47.8	27.4	41.2	22.7	34.1	17.5	26.3	12.0	18.1
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	34.8	52.3	28.0	42.1	24.2	36.3	20.0	30.1	15.4	23.2	10.6	16.0
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	1310		1140		1040		924		778		608	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	971		845		768		680		570		444	
r_{mx}/r_{my}		1.16		1.16		1.16		1.17		1.17		1.17	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

5

COMPOSITE
HSS6

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS6×4×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		28.3		22.3		19.1		15.6		12.0		8.15	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	214	321	179	269	160	240	140	211	119	179	97.5	146
	1	213	319	178	267	160	239	140	210	119	178	97.0	146
	2	210	315	176	264	157	236	138	207	117	176	95.7	144
	3	205	307	172	258	154	231	135	203	115	172	93.5	140
	4	198	297	167	250	149	224	131	196	111	167	90.5	136
	5	190	285	160	240	143	215	126	189	107	160	86.8	130
	6	180	271	152	228	137	205	120	180	102	153	82.5	124
	7	170	255	144	215	129	193	113	170	96.1	144	77.7	117
	8	158	237	134	201	121	181	106	159	90.0	135	72.5	109
	9	146	219	124	186	112	168	98.5	148	83.5	125	67.0	100
	10	133	200	114	171	103	154	90.6	136	76.8	115	61.3	92.0
	11	121	181	104	156	93.8	141	82.7	124	70.0	105	55.6	83.4
	12	108	163	93.6	140	84.7	127	74.8	112	63.3	94.9	50.0	75.0
	13	96.3	145	83.6	125	75.8	114	67.0	101	56.7	85.1	44.5	66.8
	14	84.8	127	74.0	111	67.3	101	59.5	89.3	50.3	75.5	39.3	58.9
	15	73.9	111	64.8	97.2	59.0	88.5	52.3	78.5	44.2	66.3	34.3	51.4
	16	65.0	97.5	57.0	85.5	51.9	77.8	46.0	69.0	38.9	58.3	30.1	45.2
	17	57.6	86.3	50.5	75.7	46.0	68.9	40.7	61.1	34.4	51.6	26.7	40.1
	18	51.3	77.0	45.0	67.5	41.0	61.5	36.3	54.5	30.7	46.1	23.8	35.7
	19	46.1	69.1	40.4	60.6	36.8	55.2	32.6	48.9	27.6	41.3	21.4	32.1
	20	41.6	62.4	36.5	54.7	33.2	49.8	29.4	44.1	24.9	37.3	19.3	28.9
	21	37.7	56.6	33.1	49.6	30.1	45.2	26.7	40.0	22.6	33.8	17.5	26.2
	22	34.4	51.6	30.1	45.2	27.4	41.2	24.3	36.5	20.6	30.8	15.9	23.9
	23	31.4	47.2	27.6	41.4	25.1	37.7	22.3	33.4	18.8	28.2	14.6	21.9
	24	28.9	43.3	25.3	38.0	23.1	34.6	20.4	30.7	17.3	25.9	13.4	20.1
	25	26.6	39.9	23.3	35.0	21.2	31.9	18.8	28.3	15.9	23.9	12.3	18.5
	26					19.6	29.5	17.4	26.1	14.7	22.1	11.4	17.1
	27									13.6	20.5	10.6	15.9
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	33.6	50.5	27.3	41.0	23.6	35.4	19.6	29.4	15.2	22.8	10.5	15.7
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	25.2	37.9	20.5	30.8	17.8	26.7	14.8	22.2	11.5	17.3	7.94	11.9
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	1070		942		860		765		651		508	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	546		480		436		386		327		253	
r_{mx}/r_{my}		1.40		1.40		1.40		1.41		1.41		1.42	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

5
COMPOSITE
HSS6

Shape		HSS6×3×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		24.9		19.7		16.9		13.9		10.7		7.30	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	182	273	152	228	135	203	118	177	99.3	149	79.9	120
	1	180	270	151	226	134	201	117	176	98.5	148	79.3	119
	2	176	264	147	221	131	197	114	172	96.3	144	77.5	116
	3	168	253	141	212	126	189	110	165	92.8	139	74.5	112
	4	159	238	134	201	120	179	105	157	88.1	132	70.6	106
	5	147	221	125	187	112	167	97.6	146	82.3	123	65.8	98.8
	6	134	201	114	171	103	154	89.9	135	75.8	114	60.5	90.7
	7	120	180	103	154	92.8	139	81.5	122	68.8	103	54.7	82.0
	8	106	159	91.5	137	82.6	124	72.7	109	61.5	92.3	48.7	73.0
	9	91.8	138	79.9	120	72.5	109	64.0	96.0	54.2	81.2	42.7	64.0
	10	78.2	117	68.8	103	62.6	93.9	55.4	83.1	47.0	70.5	36.9	55.3
	11	65.4	98.0	58.2	87.3	53.3	79.9	47.3	70.9	40.2	60.2	31.3	47.0
	12	54.9	82.4	48.9	73.3	44.8	67.1	39.8	59.7	33.8	50.8	26.3	39.5
	13	46.8	70.2	41.7	62.5	38.1	57.2	33.9	50.9	28.8	43.2	22.4	33.6
	14	40.3	60.5	35.9	53.9	32.9	49.3	29.2	43.9	24.9	37.3	19.3	29.0
	15	35.1	52.7	31.3	46.9	28.6	43.0	25.5	38.2	21.7	32.5	16.8	25.3
	16	30.9	46.3	27.5	41.2	25.2	37.8	22.4	33.6	19.0	28.5	14.8	22.2
	17	27.4	41.0	24.4	36.5	22.3	33.5	19.8	29.8	16.9	25.3	13.1	19.7
	18	24.4	36.6	21.7	32.6	19.9	29.8	17.7	26.5	15.0	22.6	11.7	17.5
	19			19.5	29.2	17.9	26.8	15.9	23.8	13.5	20.2	10.5	15.8
	20							14.3	21.5	12.2	18.3	9.48	14.2
	21											8.60	12.9
Properties													
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	27.7	41.7	22.7	34.2	19.8	29.7	16.5	24.8	12.8	19.3	8.89	13.4
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	16.7	25.1	13.8	20.8	12.1	18.2	10.1	15.2	7.91	11.9	5.51	8.28
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	831		739		678		605		518		409	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	259		232		212		189		160		124	
r_{mx}/r_{my}		1.79		1.79		1.79		1.79		1.80		1.81	
ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.											
Ω_c = 2.00	ϕ_c = 0.75												

5

COMPOSITE HSS5

Table 4-14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS5×4×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		24.9		19.7		16.9		13.9		10.7		7.30	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL)$_y$ with respect to weak axis (ft)	0	186	279	156	234	140	209	122	183	104	155	84.1	126
	1	185	278	155	233	139	208	122	182	103	155	83.7	126
	2	182	273	153	230	137	206	120	180	102	152	82.6	124
	3	178	267	150	224	134	201	117	176	99.4	149	80.7	121
	4	172	258	145	217	130	195	114	171	96.3	144	78.0	117
	5	164	247	139	208	124	187	109	164	92.5	139	74.8	112
	6	156	234	132	198	118	178	104	156	88.1	132	71.1	107
	7	146	219	124	186	112	167	98.0	147	83.1	125	66.8	100
	8	136	203	116	173	104	156	91.6	137	77.7	116	62.3	93.4
	9	125	187	107	160	96.3	144	84.8	127	72.0	108	57.5	86.3
	10	114	170	97.7	147	88.3	132	77.8	117	66.1	99.1	52.6	78.9
	11	102	154	88.5	133	80.2	120	70.8	106	60.1	90.2	47.7	71.5
	12	91.4	137	79.5	119	72.2	108	63.8	95.7	54.2	81.4	42.8	64.2
	13	80.8	121	70.7	106	64.3	96.5	57.0	85.5	48.5	72.7	38.0	57.1
	14	70.6	106	62.3	93.5	56.9	85.3	50.5	75.7	43.0	64.4	33.5	50.3
	15	61.5	92.2	54.4	81.6	49.7	74.5	44.2	66.3	37.6	56.4	29.2	43.8
	16	54.0	81.1	47.8	71.7	43.7	65.5	38.8	58.3	33.1	49.6	25.7	38.5
	17	47.9	71.8	42.3	63.5	38.7	58.0	34.4	51.6	29.3	43.9	22.8	34.1
	18	42.7	64.0	37.8	56.6	34.5	51.8	30.7	46.0	26.1	39.2	20.3	30.4
	19	38.3	57.5	33.9	50.8	31.0	46.5	27.5	41.3	23.5	35.2	18.2	27.3
	20	34.6	51.9	30.6	45.9	28.0	41.9	24.9	37.3	21.2	31.8	16.4	24.7
	21	31.4	47.1	27.7	41.6	25.4	38.0	22.5	33.8	19.2	28.8	14.9	22.4
	22	28.6	42.9	25.3	37.9	23.1	34.7	20.5	30.8	17.5	26.2	13.6	20.4
	23	26.2	39.2	23.1	34.7	21.1	31.7	18.8	28.2	16.0	24.0	12.4	18.6
	24	24.0	36.0	21.2	31.9	19.4	29.1	17.3	25.9	14.7	22.0	11.4	17.1
	25			19.6	29.4	17.9	26.8	15.9	23.9	13.5	20.3	10.5	15.8
	26							14.7	22.1	12.5	18.8	9.73	14.6
	27											9.02	13.5

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	25.1	37.8	20.6	30.9	17.9	26.9	14.9	22.4	11.6	17.4	8.03	12.1
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	21.5	32.2	17.6	26.5	15.3	23.0	12.8	19.2	10.0	15.0	6.90	10.4
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	658		582		533		475		406		317	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	454		402		367		327		278		216	
r_{mx}/r_{my}		1.20		1.20		1.20		1.21		1.21		1.21	
ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.											
Ω_c = 2.00	ϕ_c = 0.75												

F_y = 46 ksi
f'_c = 5 ksi

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

COMPOSITE HSS5

Shape		HSS5×3×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		21.5		17.2		14.8		12.2		9.43		6.45	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	156	234	131	196	117	175	102	153	85.6	128	68.7	103
	1	155	232	130	195	116	174	101	152	85.0	127	68.1	102
	2	151	226	127	190	113	170	98.7	148	83.0	125	66.5	99.8
	3	144	216	122	182	109	163	95.0	142	79.9	120	64.0	96.0
	4	135	203	115	172	103	154	89.9	135	75.7	114	60.6	90.9
	5	125	188	107	160	95.7	144	83.9	126	70.7	106	56.4	84.7
	6	114	170	97.4	146	87.7	132	77.0	115	65.0	97.5	51.8	77.7
	7	101	152	87.5	131	79.1	119	69.6	104	58.8	88.2	46.8	70.1
	8	88.9	133	77.4	116	70.2	105	61.9	92.9	52.4	78.6	41.6	62.3
	9	76.5	115	67.3	101	61.3	92.0	54.3	81.4	46.0	69.0	36.4	54.6
	10	64.8	97.1	57.6	86.4	52.7	79.1	46.8	70.2	39.8	59.7	31.3	47.0
	11	53.8	80.7	48.4	72.6	44.5	66.8	39.8	59.6	33.9	50.8	26.6	39.8
	12	45.2	67.8	40.7	61.0	37.4	56.1	33.4	50.1	28.5	42.7	22.3	33.5
	13	38.5	57.8	34.7	52.0	31.9	47.8	28.5	42.7	24.3	36.4	19.0	28.5
	14	33.2	49.8	29.9	44.8	27.5	41.2	24.5	36.8	20.9	31.4	16.4	24.6
	15	28.9	43.4	26.0	39.0	23.9	35.9	21.4	32.1	18.2	27.3	14.3	21.4
	16	25.4	38.2	22.9	34.3	21.0	31.6	18.8	28.2	16.0	24.0	12.5	18.8
	17	22.5	33.8	20.3	30.4	18.6	28.0	16.6	25.0	14.2	21.3	11.1	16.7
	18	20.1	30.2	18.1	27.1	16.6	24.9	14.8	22.3	12.7	19.0	9.92	14.9
	19			16.2	24.3	14.9	22.4	13.3	20.0	11.4	17.0	8.90	13.3
	20									10.3	15.4	8.03	12.0

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	20.3	30.5	16.8	25.3	14.7	22.1	12.4	18.6	9.66	14.5	6.73	10.1
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	14.0	21.1	11.7	17.6	10.3	15.4	8.65	13.0	6.79	10.2	4.74	7.13
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	501		451		415		372		320		252	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	215		193		177		158		135		106	
r_{mx}/r_{my}			1.53		1.53		1.53		1.53		1.54		1.55	

ASD	LRFD	
Ω_c = 2.00	ϕ_c = 0.75	Note: Heavy line indicates *Kl/r* equal to or greater than 200.

COMPOSITE
HSS5–HSS4

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS5×2½×						HSS4×3×					
		¼		3/16		⅛		⅜		5/16		¼	
t_{design}, in.		0.233		0.174		0.116		0.349		0.291		0.233	
Steel Wt/ft		11.3		8.79		6.02		14.6		12.7		10.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	91.7	138	76.7	115	60.9	91.4	110	165	98.3	147	85.7	129
	1	90.7	136	75.8	114	60.3	90.4	109	163	97.5	146	85.1	128
	2	87.7	132	73.4	110	58.3	87.5	106	159	95.1	143	83.0	125
	3	83.0	124	69.6	104	55.2	82.9	102	153	91.2	137	79.7	120
	4	76.8	115	64.5	96.7	51.2	76.8	95.7	144	86.0	129	75.3	113
	5	69.5	104	58.5	87.8	46.4	69.6	88.6	133	79.8	120	70.0	105
	6	61.6	92.4	51.9	77.9	41.2	61.8	80.5	121	72.8	109	64.1	96.1
	7	53.3	80.0	45.1	67.7	35.7	53.6	72.0	108	65.3	98.0	57.7	86.5
	8	45.2	67.8	38.4	57.6	30.3	45.5	63.3	94.9	57.7	86.5	51.1	76.6
	9	37.5	56.2	31.9	47.9	25.2	37.8	54.6	81.9	50.0	75.1	44.5	66.8
	10	30.5	45.7	26.0	39.0	20.5	30.8	46.4	69.5	42.7	64.1	38.2	57.2
	11	25.2	37.8	21.5	32.3	17.0	25.5	38.6	57.9	35.8	53.7	32.1	48.2
	12	21.1	31.7	18.1	27.1	14.3	21.4	32.5	48.7	30.1	45.1	27.0	40.5
	13	18.0	27.0	15.4	23.1	12.1	18.2	27.7	41.5	25.6	38.4	23.0	34.5
	14	15.5	23.3	13.3	19.9	10.5	15.7	23.8	35.8	22.1	33.1	19.8	29.8
	15	13.5	20.3	11.6	17.4	9.12	13.7	20.8	31.2	19.2	28.9	17.3	25.9
	16	11.9	17.8	10.2	15.2	8.02	12.0	18.3	27.4	16.9	25.4	15.2	22.8
	17			9.01	13.5	7.10	10.7	16.2	24.3	15.0	22.5	13.5	20.2
	18							14.4	21.6	13.4	20.0	12.0	18.0
	19											10.8	16.2

Properties

M_{nx}/Ω_b	$\phi_b M_{nx}$	kip-ft	11.1	16.7	8.70	13.1	6.08	9.14	11.7	17.7	10.4	15.6	8.76	13.2
M_{ny}/Ω_b	$\phi_b M_{ny}$	kip-ft	6.78	10.2	5.35	8.04	3.76	5.65	9.58	14.4	8.47	12.7	7.17	10.8
$P_{ex}(K_xL_x)^2/10^4$		kip-in.^{2}	321		275		220		248		229		206	
$P_{ey}(K_yL_y)^2/10^4$		kip-in.^{2}	100		85.5		67.4		154		142		128	
r_{mx}/r_{my}			1.79		1.79		1.81		1.27		1.27		1.27	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4-14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

COMPOSITE
HSS4

Shape		HSS4×3×				HSS4×2½×							
		3/16		1/8		3/8		5/16		1/4		3/16	
t_{design}, in.		0.174		0.116		0.349		0.291		0.233		0.174	
Steel Wt/ft		8.15		5.60		13.4		11.6		9.63		7.51	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$ with respect to weak axis (ft)	0	72.0	108	57.5	86.2	98.4	148	88	132	76.6	115	64.1	96.1
	1	71.4	107	57.0	85.5	97.1	146	86.9	130	75.8	114	63.4	95.1
	2	69.7	105	55.6	83.5	93.6	140	83.9	126	73.2	110	61.3	91.9
	3	67.0	101	53.5	80.2	87.9	132	79.0	119	69.1	104	58.0	86.9
	4	63.4	95.1	50.5	75.8	80.6	121	72.7	109	63.8	95.6	53.6	80.4
	5	59.0	88.5	47.0	70.5	72.1	108	65.3	98.0	57.5	86.2	48.5	72.7
	6	54.1	81.1	43.1	64.6	62.9	94.4	57.3	85.9	50.7	76.0	42.9	64.3
	7	48.8	73.2	38.8	58.2	53.5	80.3	49.1	73.6	43.6	65.5	37.1	55.6
	8	43.3	65.0	34.4	51.6	44.4	66.7	41.0	61.6	36.7	55.1	31.3	47.0
	9	37.9	56.8	30.0	45.1	35.9	53.9	33.5	50.2	30.2	45.3	25.9	38.9
	10	32.6	48.8	25.8	38.7	29.1	43.6	27.1	40.7	24.5	36.7	21.0	31.6
	11	27.5	41.3	21.8	32.7	24.0	36.1	22.4	33.6	20.2	30.3	17.4	26.1
	12	23.1	34.7	18.3	27.4	20.2	30.3	18.8	28.2	17.0	25.5	14.6	21.9
	13	19.7	29.6	15.6	23.4	17.2	25.8	16.0	24.1	14.5	21.7	12.4	18.7
	14	17.0	25.5	13.4	20.2	14.8	22.3	13.8	20.7	12.5	18.7	10.7	16.1
	15	14.8	22.2	11.7	17.6	12.9	19.4	12.0	18.1	10.9	16.3	9.35	14.0
	16	13.0	19.5	10.3	15.4					9.56	14.3	8.22	12.3
	17	11.5	17.3	9.12	13.7								
	18	10.3	15.4	8.13	12.2								
	19	9.23	13.8	7.30	10.9								
	20			6.59	9.88								

Properties

		HSS4×3×3/16		HSS4×3×1/8		HSS4×2½×3/8		HSS4×2½×5/16		HSS4×2½×1/4		HSS4×2½×3/16	
M_{nx}/Ω_b	$\phi_b M_{nx}$ kip-ft	6.90	10.4	4.84	7.27	10.3	15.5	9.12	13.7	7.75	11.6	6.13	9.22
M_{ny}/Ω_b	$\phi_b M_{ny}$ kip-ft	5.66	8.50	3.97	5.97	7.34	11.0	6.53	9.82	5.57	8.37	4.42	6.65
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	177		141		209		194		176		153	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	109		86.5		95.5		89.0		80.4		69.1	
r_{mx}/r_{my}		1.27		1.28		1.48		1.48		1.48		1.49	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

5

COMPOSITE
HSS4

Table 4–14 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Rectangular HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS4×3×		HSS4×2×									
		1/8		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.116		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		5.17		12.1		10.5		8.78		6.87		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length $(KL)_y$, with respect to weak axis (ft)	0	50.8	76.2	86.8	130	77.7	116	67.5	101	56.2	84.3	44.1	66.2
	1	50.2	75.3	85.1	128	76.2	114	66.3	99.5	55.3	82.9	43.4	65.1
	2	48.6	72.9	80.2	120	72.1	108	62.9	94.3	52.5	78.7	41.2	61.9
	3	45.9	68.9	72.7	109	65.6	98.5	57.5	86.3	48.2	72.3	37.9	56.9
	4	42.5	63.7	63.4	95.1	57.6	86.4	50.8	76.2	42.7	64.1	33.7	50.5
	5	38.4	57.7	53.1	79.7	48.7	73.0	43.2	64.9	36.6	54.9	29.0	43.4
	6	34.0	51.0	42.8	64.2	39.7	59.5	35.6	53.3	30.3	45.5	24.1	36.1
	7	29.4	44.2	33.1	49.6	31.1	46.7	28.2	42.3	24.3	36.4	19.3	29.0
	8	24.9	37.4	25.3	38.0	23.9	35.8	21.7	32.6	18.8	28.2	15.0	22.6
	9	20.6	30.9	20.0	30.0	18.9	28.3	17.2	25.8	14.9	22.3	11.9	17.8
	10	16.8	25.1	16.2	24.3	15.3	22.9	13.9	20.9	12.0	18.1	9.62	14.4
	11	13.8	20.8	13.4	20.1	12.6	18.9	11.5	17.2	10.0	14.9	7.95	11.9
	12	11.6	17.5	11.3	16.9	10.6	15.9	9.66	14.5	8.36	12.5	6.68	10.0
	13	9.91	14.9							7.12	10.7	5.69	8.54
	14	8.55	12.8										
	15	7.45	11.2										
	16	6.54	9.82										
	17	5.80	8.70										
Properties													
M_{nx}/Ω_b / $\phi_b M_{nx}$	kip-ft	4.32	6.49	8.82	13.3	7.88	11.8	6.74	10.1	5.37	8.07	3.80	5.71
M_{ny}/Ω_b / $\phi_b M_{ny}$	kip-ft	3.13	4.70	5.30	7.96	4.76	7.16	4.10	6.17	3.29	4.94	2.34	3.52
$P_{ex}(K_xL_x)^2/10^4$	kip-in.^{2}	122		170		160		145		127		103	
$P_{ey}(K_yL_y)^2/10^4$	kip-in.^{2}	55.0		53.3		50.1		45.6		39.5		31.5	
r_{mx}/r_{my}		1.49		1.79		1.79		1.78		1.79		1.80	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–15
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

4

COMPOSITE
HSS16–HSS14

Shape		HSS16×16×						HSS14×14×					
		1/2		3/8		5/16		5/8		1/2		3/8	
t_{design}, in.		0.465		0.349		0.291		0.581		0.465		0.349	
Steel Wt/ft		103		78.4		65.8		110		89.6		68.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	1040	1560	893	1340	820	1230	977	1470	856	1280	732	1100
	6	1030	1540	884	1330	811	1220	964	1450	845	1270	722	1080
	7	1020	1530	881	1320	808	1210	960	1440	841	1260	718	1080
	8	1020	1530	877	1320	804	1210	955	1430	837	1250	714	1070
	9	1010	1520	872	1310	800	1200	949	1420	831	1250	710	1060
	10	1010	1510	868	1300	795	1190	942	1410	826	1240	705	1060
	11	1000	1500	862	1290	790	1190	935	1400	819	1230	699	1050
	12	996	1490	857	1280	785	1180	927	1390	813	1220	693	1040
	13	989	1480	850	1280	779	1170	919	1380	805	1210	687	1030
	14	982	1470	844	1270	773	1160	910	1370	797	1200	680	1020
	15	974	1460	837	1250	766	1150	901	1350	789	1180	673	1010
	16	965	1450	829	1240	759	1140	891	1340	780	1170	665	997
	17	956	1430	821	1230	752	1130	880	1320	771	1160	657	985
	18	947	1420	813	1220	744	1120	869	1300	761	1140	648	972
	19	937	1410	804	1210	736	1100	857	1290	751	1130	639	959
	20	927	1390	795	1190	727	1090	845	1270	740	1110	630	945
	21	916	1370	786	1180	719	1080	833	1250	729	1090	620	931
	22	905	1360	776	1160	709	1060	820	1230	718	1080	611	916
	23	894	1340	766	1150	700	1050	807	1210	706	1060	600	901
	24	882	1320	755	1130	690	1040	793	1190	694	1040	590	885
	25	870	1310	745	1120	680	1020	779	1170	682	1020	579	869
	26	858	1290	734	1100	670	1000	765	1150	669	1000	568	852
	27	845	1270	723	1080	659	989	750	1130	656	985	557	836
	28	832	1250	711	1070	649	973	736	1100	643	965	546	819
	29	819	1230	699	1050	638	957	721	1080	630	945	534	801
	30	805	1210	687	1030	627	940	705	1060	617	925	523	784
	32	778	1170	663	995	604	906	674	1010	589	884	499	748
	34	749	1120	638	957	581	871	643	964	562	843	475	712
	36	720	1080	613	919	557	835	611	917	534	801	451	676
	38	691	1040	587	880	533	799	579	869	506	759	426	639
	40	661	992	561	841	509	763	547	821	478	717	402	603

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	376	566	289	435	243	366	347	521	285	428	219	329
$P_e(KL)^2/10^4$		kip-in.^{2}	44400		37000		33100		32500		28400		23600	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

4

COMPOSITE
HSS14–HSS12

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS14×14×		HSS12×12×									
		5/16		5/8		1/2		3/8		5/16		1/4	
t_{design}, in.		0.291		0.581		0.465		0.349		0.291		0.233	
Steel Wt/ft		57.3		93.1		75.9		58.0		48.8		39.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	668	1000	790	1180	689	1030	584	876	530	795	475	713
	6	659	988	776	1160	677	1010	573	860	520	780	466	699
	7	656	983	771	1160	672	1010	570	855	517	775	463	695
	8	652	978	765	1150	667	1000	565	848	513	769	459	689
	9	648	971	758	1140	662	993	561	841	508	763	455	683
	10	643	964	751	1130	656	983	555	833	504	755	451	676
	11	638	957	743	1120	649	973	549	824	498	747	446	669
	12	632	948	735	1100	642	962	543	815	492	739	440	661
	13	626	939	726	1090	634	950	536	805	486	729	435	652
	14	620	930	716	1070	625	938	529	794	479	719	429	643
	15	613	919	706	1060	616	925	522	782	472	709	422	633
	16	606	909	695	1040	607	911	513	770	465	698	415	623
	17	598	897	684	1030	597	896	505	758	457	686	408	612
	18	590	885	672	1010	587	881	496	744	449	674	401	601
	19	582	873	660	990	576	865	487	731	441	661	393	589
	20	573	860	647	971	566	848	478	717	432	648	385	578
	21	564	847	634	951	554	831	468	702	423	635	377	565
	22	555	833	621	931	543	814	458	687	414	621	368	553
	23	546	819	607	910	531	796	448	672	405	607	360	540
	24	536	804	593	889	519	778	437	656	395	592	351	526
	25	526	789	579	868	506	759	427	640	385	578	342	513
	26	516	774	564	846	494	740	416	624	375	563	333	500
	27	506	758	549	824	481	721	405	608	365	548	324	486
	28	495	743	535	802	468	702	394	591	355	533	315	472
	29	484	727	520	780	455	683	383	574	345	517	305	458
	30	474	710	505	757	442	663	372	558	335	502	296	444
	32	452	677	475	712	416	624	349	524	314	471	277	416
	34	429	644	444	667	390	585	327	490	294	441	259	388
	36	407	611	414	622	364	546	305	457	273	410	240	361
	38	385	577	385	578	338	507	283	424	254	380	222	334
	40	362	544	356	535	313	470	262	392	234	351	205	307
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	185	278	250	376	206	309	159	239	134	202	109	164
$P_e(KL)^2/10^4$	kip-in.^{2}	21000		19200		16800		14000		12500		10900	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 46 ksi
f'_c = 4 ksi

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

4
COMPOSITE
HSS10

Shape		HSS10×10×											
		5/8		1/2		3/8		5/16		1/4		3/16	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		76.1		62.3		47.8		40.3		32.6		24.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	616	924	535	802	450	674	406	608	361	541	314	471
	6	600	900	521	782	438	657	395	593	351	527	305	458
	7	594	891	516	774	434	651	391	587	348	522	302	453
	8	588	882	511	766	430	644	387	581	344	516	299	448
	9	581	871	505	757	424	637	383	574	340	509	295	442
	10	573	859	498	747	419	628	377	566	335	502	291	436
	11	564	846	490	736	412	619	372	557	330	495	286	429
	12	555	832	482	724	406	609	365	548	324	486	281	421
	13	545	817	474	711	399	598	359	538	318	477	276	413
	14	534	801	465	697	391	587	352	528	312	468	270	405
	15	523	784	455	683	383	575	345	517	305	458	264	396
	16	511	767	445	668	375	562	337	506	298	448	258	387
	17	499	749	435	652	366	549	329	494	291	437	251	377
	18	487	730	424	636	357	535	321	481	284	426	245	367
	19	474	710	413	620	348	522	312	469	276	414	238	357
	20	460	691	402	603	338	507	304	456	268	402	231	346
	21	447	670	390	585	328	493	295	442	260	390	224	335
	22	433	650	378	568	319	478	286	429	252	378	216	324
	23	419	629	366	550	308	463	277	415	244	366	209	313
	24	405	608	354	532	298	447	267	401	235	353	201	302
	25	391	586	342	513	288	432	258	387	227	341	194	291
	26	377	565	330	495	278	417	249	373	219	328	187	280
	27	362	544	318	477	268	401	239	359	210	315	179	269
	28	348	522	305	458	257	386	230	345	202	303	172	258
	29	334	501	293	440	247	371	221	331	194	290	164	246
	30	320	480	281	422	237	355	212	317	185	278	157	236
	32	292	439	257	386	217	325	193	290	169	253	143	214
	34	266	398	234	351	197	296	176	264	153	230	129	193
	36	240	360	212	318	179	268	159	238	138	207	116	173
	38	215	323	190	286	161	241	143	214	124	186	104	156
	40	194	291	172	258	145	217	129	193	112	168	93.6	140
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	168	252	139	210	108	163	92.0	138	75.0	113	57.0	85.6
$P_e(KL)^2/10^4$	kip-in.^{2}	10200		9030		7620		6770		5880		4910	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

4

COMPOSITE
HSS9

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS9×9×											
		5/8		1/2		3/8		5/16		1/4		3/16	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		67.6		55.5		42.7		36.0		29.2		22.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	534	801	463	694	388	581	348	523	308	463	267	400
	6	517	775	448	672	376	563	338	506	299	448	258	387
	7	511	766	443	665	371	557	334	501	295	443	255	382
	8	504	756	437	656	366	550	329	494	291	437	251	377
	9	496	744	431	646	361	542	324	487	287	430	247	371
	10	488	732	423	635	355	533	319	479	282	423	243	364
	11	479	718	416	624	349	523	313	470	277	415	238	357
	12	469	703	407	611	342	513	307	460	271	406	233	350
	13	458	687	398	598	334	501	300	450	265	397	228	342
	14	447	671	389	583	327	490	293	440	259	388	222	333
	15	435	653	379	569	318	478	286	428	252	378	216	324
	16	423	635	369	553	310	465	278	417	245	367	210	315
	17	411	616	358	537	301	452	270	405	238	357	204	305
	18	398	597	347	521	292	438	262	393	230	346	197	296
	19	385	577	336	504	283	424	253	380	223	334	190	286
	20	372	557	325	487	273	410	245	367	215	323	184	275
	21	358	537	313	470	264	395	236	354	207	311	177	265
	22	344	517	301	452	254	381	227	341	199	299	170	255
	23	331	496	290	434	244	366	218	328	191	287	163	244
	24	317	475	278	417	234	351	209	314	184	275	156	234
	25	303	455	266	399	224	337	201	301	176	263	149	223
	26	289	434	254	381	215	322	192	288	168	252	142	213
	27	276	414	243	364	205	307	183	274	160	240	135	202
	28	262	394	231	347	195	293	174	261	152	228	128	192
	29	249	374	220	329	186	279	166	249	145	217	122	182
	30	236	354	208	313	176	265	157	236	137	206	115	173
	32	211	317	187	280	158	237	141	211	123	184	102	153
	34	187	281	166	249	141	211	125	188	109	163	90.6	136
	36	167	250	148	222	126	188	112	168	96.9	145	80.8	121
	38	150	225	133	199	113	169	100	150	87.0	131	72.6	109
	40	135	203	120	180	102	153	90.5	136	78.5	118	65.5	98.2
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	133	200	111	167	86.8	130	73.8	111	60.2	90.5	45.9	69.0
$P_e(KL)^2/10^4$	kip-in.^{2}	7100		6290		5340		4750		4130		3440	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

F_y = 46 ksi
f'_c = 4 ksi

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

4
COMPOSITE
HSS8

Shape		HSS8×8×									
		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233	
Steel Wt/ft		59.1		48.7		37.6		31.8		25.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length (*KL*) (ft)	0	456	683	394	591	329	493	295	442	260	390
	6	437	655	378	568	316	474	283	425	249	374
	7	430	645	373	559	312	467	279	419	246	369
	8	423	634	367	550	306	460	275	412	242	362
	9	414	622	360	539	301	451	269	404	237	356
	10	405	608	352	528	294	442	264	396	232	348
	11	396	593	344	516	288	431	258	387	227	340
	12	385	578	335	502	280	421	251	377	221	331
	13	374	561	326	488	273	409	244	366	215	322
	14	362	544	316	474	265	397	237	356	208	312
	15	350	525	306	458	256	385	230	344	201	302
	16	338	507	295	442	248	372	222	333	195	292
	17	325	488	284	426	239	358	214	321	187	281
	18	312	468	273	410	230	345	206	309	180	270
	19	299	448	262	393	220	331	197	296	173	259
	20	286	428	251	376	211	317	189	284	165	248
	21	272	408	239	359	202	303	181	271	158	237
	22	259	388	228	342	192	289	172	258	150	226
	23	246	368	216	325	183	275	164	246	143	214
	24	233	349	205	308	174	261	155	233	136	203
	25	220	329	194	291	165	247	147	221	128	192
	26	207	310	183	275	156	233	139	209	121	182
	27	194	292	173	259	147	220	131	197	114	171
	28	182	274	162	243	138	207	123	185	107	161
	29	170	255	152	228	130	194	116	174	100	151
	30	159	239	142	213	121	182	108	162	93.9	141
	32	140	210	125	187	106	160	95.1	143	82.5	124
	34	124	186	110	166	94.3	141	84.3	126	73.1	110
	36	111	166	98.6	148	84.1	126	75.2	113	65.2	97.8
	38	99.2	149	88.5	133	75.5	113	67.4	101	58.5	87.8
	40	89.5	134	79.8	120	68.1	102	60.9	91.3	52.8	79.2

Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	103	154	86.0	129	67.6	102	57.6	86.6	47.1	70.9
$P_e(KL)^2/10^4$ kip-in.^{2}		4710		4190		3590		3200		2780	
ASD	**LRFD**										
$\Omega_c = 2.00$	$\phi_c = 0.75$										

4

COMPOSITE
HSS8–HSS7

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS8×8×		HSS7×7×							
		3/16		5/8		1/2		3/8		5/16	
t_{design}, in.		0.174		0.581		0.465		0.349		0.291	
Steel Wt/ft		19.6		50.6		41.9		32.5		27.5	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	223	334	381	571	329	494	274	411	244	367
	6	214	321	360	540	312	468	260	389	232	348
	7	211	316	353	529	306	459	255	382	228	342
	8	207	310	344	517	299	448	249	374	223	334
	9	203	304	335	503	291	437	243	365	217	326
	10	198	298	326	488	283	425	236	355	212	317
	11	194	291	315	473	274	411	229	344	205	308
	12	189	283	304	456	265	397	222	332	198	298
	13	183	275	292	439	255	383	214	321	191	287
	14	177	266	280	421	245	367	205	308	184	276
	15	172	257	268	402	235	352	197	295	176	265
	16	166	248	255	383	224	336	188	282	169	253
	17	159	239	243	364	213	319	179	269	161	241
	18	153	229	230	345	202	303	170	255	153	229
	19	146	220	217	325	191	287	161	242	145	217
	20	140	210	204	306	180	270	152	229	137	205
	21	133	200	191	287	169	254	144	215	129	194
	22	127	190	179	269	159	238	135	202	121	182
	23	120	181	167	250	148	223	126	189	114	170
	24	114	171	155	233	138	207	118	177	106	159
	25	108	162	143	215	128	193	110	164	98.9	148
	26	102	152	133	199	119	178	102	152	91.7	137
	27	95.4	143	123	185	110	165	94.1	141	85.0	127
	28	89.5	134	114	172	102	153	87.5	131	79.0	119
	29	83.6	125	107	160	95.4	143	81.6	122	73.7	111
	30	78.1	117	99.6	149	89.1	134	76.2	114	68.8	103
	32	68.7	103	87.6	131	78.3	118	67.0	101	60.5	90.8
	34	60.8	91.2	77.6	116	69.4	104	59.4	89.0	53.6	80.4
	36	54.3	81.4	69.2	104	61.9	92.8	52.9	79.4	47.8	71.7
	38	48.7	73.0	62.1	93.1	55.6	83.3	47.5	71.3	42.9	64.4
	40	43.9	65.9	56.0	84.1	50.1	75.2	42.9	64.3	38.7	58.1
Properties											
M_n/Ω_b | $\phi_b M_n$	kip-ft	36.0	54.1	75.9	114	64.1	96.4	50.7	76.2	43.4	65.2
$P_e(KL)^2/10^4$	kip-in.^{2}	2300		2940		2640		2250		2030	
ASD	LRFD										
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 46 ksi
f'_c = 4 ksi

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

Shape		HSS7×7×						HSS6×6×			
		1/4		3/16		1/8		5/8		1/2	
t_{design}, in.		0.233		0.174		0.116		0.581		0.465	
Steel Wt/ft		22.4		17.1		11.6		42.1		35.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	214	322	183	274	151	226	309	463	267	401
	6	203	305	173	260	142	213	286	428	248	372
	7	200	299	170	255	139	209	278	416	241	362
	8	195	293	166	249	136	204	269	403	234	351
	9	190	286	162	243	132	199	259	388	226	338
	10	185	278	157	236	129	193	248	372	217	325
	11	180	270	152	229	124	186	237	356	207	311
	12	174	261	147	221	120	180	225	338	198	297
	13	168	251	142	213	115	173	213	320	188	281
	14	161	242	136	204	110	166	201	302	177	266
	15	154	232	130	196	105	158	189	283	167	250
	16	148	221	125	187	100	151	176	265	156	235
	17	141	211	119	178	95.3	143	164	246	146	219
	18	134	200	112	169	90.2	135	152	228	136	203
	19	127	190	106	160	85.1	128	140	210	125	188
	20	120	179	100	151	80.0	120	129	193	116	173
	21	113	169	94.4	142	75.0	112	118	176	106	159
	22	106	159	88.5	133	70.1	105	107	161	96.8	145
	23	99.1	149	82.8	124	65.2	97.9	98.0	147	88.5	133
	24	92.5	139	77.1	116	60.6	90.9	90.0	135	81.3	122
	25	86.1	129	71.7	108	55.9	83.9	82.9	124	74.9	112
	26	79.8	120	66.3	99.4	51.7	77.6	76.7	115	69.3	104
	27	74.0	111	61.5	92.2	48.0	71.9	71.1	107	64.2	96.4
	28	68.8	103	57.1	85.7	44.6	66.9	66.1	99.2	59.7	89.6
	29	64.1	96.2	53.3	79.9	41.6	62.4	61.6	92.5	55.7	83.5
	30	59.9	89.9	49.8	74.7	38.8	58.3	57.6	86.4	52.0	78.1
	32	52.7	79.0	43.8	65.6	34.1	51.2	50.6	75.9	45.7	68.6
	34	46.7	70.0	38.8	58.1	30.2	45.4	44.8	67.3	40.5	60.8
	36	41.6	62.4	34.6	51.9	27.0	40.5	40.0	60.0	36.1	54.2
	38	37.4	56.0	31.0	46.5	24.2	36.3				
	40	33.7	50.6	28.0	42.0	21.9	32.8				

Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	35.6	53.6	27.3	41.0	18.7	28.0	53.2	80.0	45.4	68.3
$P_e(KL)^2/10^4$	kip-in.^{2}	1770		1470		1150		1700		1540	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

4

COMPOSITE
HSS6

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS6×6×									
		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		27.4		23.3		19.0		14.5		9.85	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	222	333	198	296	172	259	146	219	119	178
	6	206	309	184	276	161	241	136	203	110	165
	7	201	301	179	269	157	235	132	198	107	160
	8	195	292	174	261	152	228	128	192	104	155
	9	188	283	168	252	147	220	124	186	100	150
	10	181	272	162	243	142	212	119	179	96.1	144
	11	174	261	155	233	136	204	114	171	91.9	138
	12	166	249	148	223	130	195	109	164	87.6	131
	13	158	237	141	212	124	185	104	156	83.1	125
	14	149	224	134	201	117	176	98.4	148	78.5	118
	15	141	211	126	189	111	166	92.8	139	73.9	111
	16	132	198	119	178	104	156	87.2	131	69.2	104
	17	124	186	111	167	97.6	146	81.6	122	64.6	96.8
	18	115	173	104	156	91.1	137	76.1	114	60.0	90.0
	19	107	161	96.4	145	84.7	127	70.6	106	55.5	83.2
	20	98.9	148	89.2	134	78.4	118	65.3	98.0	51.1	76.7
	21	91.1	137	82.2	123	72.3	108	60.2	90.3	46.9	70.3
	22	83.3	125	75.4	113	66.4	99.6	55.1	82.6	42.8	64.1
	23	76.2	114	68.9	103	60.7	91.1	50.4	75.6	39.1	58.7
	24	70.0	105	63.3	95.0	55.8	83.7	46.3	69.4	35.9	53.9
	25	64.5	96.8	58.4	87.5	51.4	77.1	42.7	64.0	33.1	49.7
	26	59.7	89.5	54.0	80.9	47.5	71.3	39.4	59.2	30.6	45.9
	27	55.3	83.0	50.0	75.0	44.1	66.1	36.6	54.9	28.4	42.6
	28	51.4	77.2	46.5	69.8	41.0	61.5	34.0	51.0	26.4	39.6
	29	48.0	71.9	43.4	65.1	38.2	57.3	31.7	47.6	24.6	36.9
	30	44.8	67.2	40.5	60.8	35.7	53.5	29.6	44.4	23.0	34.5
	32	39.4	59.1	35.6	53.4	31.4	47.1	26.0	39.1	20.2	30.3
	34	34.9	52.3	31.6	47.3	27.8	41.7	23.1	34.6	17.9	26.9
	36	31.1	46.7	28.1	42.2	24.8	37.2	20.6	30.9	16.0	24.0
	38	27.9	41.9	25.3	37.9	22.2	33.4	18.5	27.7	14.3	21.5
Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	36.3	54.6	31.2	46.9	25.7	38.7	19.8	29.8	13.6	20.4
$P_e(KL)^2/10^4$	kip-in.^{2}	1320		1200		1060		876		680	
ASD	LRFD										
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 46 ksi
f'_c = 4 ksi

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

Shape		HSS5½×5½×										HSS5×5×	
		3/8		5/16		1/4		3/16		1/8		1/2	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
Steel Wt/ft		24.9		21.2		17.3		13.2		9.00		28.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	197	296	175	263	153	229	129	193	104	156	209	314
	1	197	295	175	263	152	228	128	192	104	155	208	313
	2	195	293	174	261	151	227	127	191	103	154	206	310
	3	193	289	172	258	149	224	126	189	102	152	203	305
	4	190	285	169	253	147	221	124	186	99.8	150	199	298
	5	186	278	165	248	144	216	121	182	97.6	146	193	290
	6	181	271	161	242	140	210	118	177	95.0	142	187	280
	7	175	263	156	234	136	204	114	172	92.0	138	179	269
	8	169	253	151	226	131	197	110	166	88.6	133	171	257
	9	162	243	145	217	126	189	106	159	85.0	127	162	243
	10	155	232	138	207	121	181	101	152	81.1	122	153	229
	11	147	221	132	197	115	172	96.5	145	77.0	115	143	215
	12	139	209	124	187	109	163	91.4	137	72.7	109	133	200
	13	131	196	117	176	103	154	86.1	129	68.4	103	123	185
	14	123	184	110	165	96.4	145	80.8	121	63.9	95.9	113	170
	15	114	171	103	154	90.0	135	75.4	113	59.5	89.3	104	155
	16	106	159	95.3	143	83.7	126	70.1	105	55.1	82.7	94.0	141
	17	97.8	147	88.1	132	77.5	116	64.8	97.2	50.8	76.2	84.8	127
	18	89.9	135	81.1	122	71.3	107	59.6	89.4	46.6	69.9	75.8	114
	19	82.2	123	74.2	111	65.4	98.1	54.6	81.9	42.5	63.8	68.1	102
	20	74.6	112	67.6	101	59.7	89.5	49.8	74.6	38.5	57.8	61.4	92.2
	21	67.7	102	61.3	91.9	54.1	81.2	45.1	67.7	34.9	52.4	55.7	83.6
	22	61.7	92.5	55.9	83.8	49.3	74.0	41.1	61.7	31.8	47.8	50.8	76.2
	23	56.4	84.6	51.1	76.6	45.1	67.7	37.6	56.4	29.1	43.7	46.5	69.7
	24	51.8	77.7	46.9	70.4	41.4	62.2	34.6	51.8	26.8	40.1	42.7	64.0
	25	47.7	71.6	43.3	64.9	38.2	57.3	31.8	47.8	24.7	37.0	39.3	59.0
	26	44.1	66.2	40.0	60.0	35.3	53.0	29.4	44.2	22.8	34.2	36.4	54.5
	27	40.9	61.4	37.1	55.6	32.7	49.1	27.3	41.0	21.1	31.7	33.7	50.6
	28	38.1	57.1	34.5	51.7	30.4	45.7	25.4	38.1	19.7	29.5	31.3	47.0
	29	35.5	53.2	32.1	48.2	28.4	42.6	23.7	35.5	18.3	27.5	29.2	43.8
	30	33.2	49.7	30.0	45.1	26.5	39.8	22.1	33.2	17.1	25.7	27.3	41.0

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	30.0	45.1	25.9	38.9	21.4	32.2	16.5	24.8	11.4	17.1	30.0	45.0
$P_e(KL)^2/10^4$		kip-in.^{2}	981		887		783		654		505		806	
ASD	LRFD													
$\Omega_c = 2.00$	$\phi_c = 0.75$													

4

COMPOSITE
HSS5

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS5×5×									
		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		22.3		19.0		15.6		12.0		8.15	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	173	260	154	231	134	201	112	168	90.0	135
	1	173	259	154	230	133	200	112	168	89.7	135
	2	171	257	152	228	132	198	111	166	88.9	133
	3	169	253	150	225	130	196	109	164	87.6	131
	4	165	248	147	221	128	192	107	161	85.8	129
	5	161	241	143	215	125	187	104	157	83.5	125
	6	156	234	139	208	121	181	101	152	80.9	121
	7	150	225	134	200	116	175	97.6	146	77.8	117
	8	143	215	128	192	111	167	93.5	140	74.4	112
	9	136	204	122	183	106	159	89.1	134	70.8	106
	10	129	193	115	173	101	151	84.4	127	66.9	100
	11	121	182	108	163	94.7	142	79.5	119	62.9	94.4
	12	113	170	101	152	88.7	133	74.4	112	58.8	88.1
	13	105	157	94.3	141	82.6	124	69.3	104	54.6	81.9
	14	96.9	145	87.2	131	76.5	115	64.2	96.3	50.4	75.6
	15	88.9	133	80.1	120	70.4	106	59.1	88.6	46.3	69.4
	16	81.1	122	73.2	110	64.4	96.6	54.1	81.1	42.2	63.3
	17	73.5	110	66.5	99.8	58.6	87.9	49.2	73.8	38.3	57.4
	18	66.1	99.2	60.0	90.1	53.0	79.6	44.6	66.8	34.5	51.7
	19	59.4	89.0	53.9	80.8	47.6	71.5	40.0	60.0	30.9	46.4
	20	53.6	80.3	48.6	73.0	43.0	64.5	36.1	54.2	27.9	41.9
	21	48.6	72.9	44.1	66.2	39.0	58.5	32.8	49.1	25.3	38.0
	22	44.3	66.4	40.2	60.3	35.5	53.3	29.9	44.8	23.1	34.6
	23	40.5	60.8	36.8	55.2	32.5	48.8	27.3	41.0	21.1	31.7
	24	37.2	55.8	33.8	50.7	29.9	44.8	25.1	37.6	19.4	29.1
	25	34.3	51.4	31.1	46.7	27.5	41.3	23.1	34.7	17.9	26.8
	26	31.7	47.5	28.8	43.2	25.4	38.2	21.4	32.1	16.5	24.8
	27	29.4	44.1	26.7	40.0	23.6	35.4	19.8	29.7	15.3	23.0
	28	27.3	41.0	24.8	37.2	21.9	32.9	18.4	27.6	14.2	21.4
	29	25.5	38.2	23.1	34.7	20.5	30.7	17.2	25.8	13.3	19.9
	30	23.8	35.7	21.6	32.4	19.1	28.7	16.1	24.1	12.4	18.6
Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	24.3	36.5	21.0	31.6	17.5	26.2	13.5	20.3	9.33	14.0
$P_e(KL)^2/10^4$	kip-in.^{2}	704		639		564		474		367	
ASD	LRFD										
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 46 ksi
f'_c = 4 ksi

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

4
COMPOSITE
HSS4$^1/_2$

Shape		HSS4$^1/_2$×4$^1/_2$×											
		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		24.9		19.7		16.9		13.9		10.7		7.30	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	181	272	150	226	134	200	116	174	96.7	145	76.9	115
	1	180	271	150	225	133	200	115	173	96.4	145	76.7	115
	2	178	267	148	222	132	198	114	171	95.4	143	75.8	114
	3	175	262	145	218	129	194	112	168	93.7	141	74.5	112
	4	170	255	142	213	126	189	109	164	91.4	137	72.6	109
	5	164	246	137	206	122	183	106	159	88.6	133	70.3	105
	6	157	236	132	197	117	176	102	153	85.3	128	67.6	101
	7	149	224	125	188	112	168	97.3	146	81.5	122	64.5	96.7
	8	141	211	119	178	106	159	92.3	138	77.3	116	61.1	91.6
	9	132	197	111	167	99.6	149	86.9	130	72.8	109	57.4	86.2
	10	122	183	104	155	92.9	139	81.2	122	68.1	102	53.6	80.5
	11	112	169	95.9	144	86.1	129	75.4	113	63.3	95.0	49.7	74.6
	12	103	154	88.0	132	79.2	119	69.5	104	58.4	87.6	45.8	68.7
	13	92.9	139	80.2	120	72.3	109	63.5	95.3	53.5	80.3	41.8	62.7
	14	83.5	125	72.5	109	65.6	98.4	57.7	86.6	48.7	73.0	37.9	56.9
	15	74.5	112	65.1	97.6	59.0	88.5	52.1	78.1	44.0	66.0	34.2	51.3
	16	65.8	98.7	57.9	86.9	52.7	79.1	46.7	70.0	39.5	59.2	30.6	45.8
	17	58.3	87.4	51.3	76.9	46.7	70.1	41.4	62.1	35.1	52.6	27.1	40.6
	18	52.0	78.0	45.7	68.6	41.7	62.5	36.9	55.4	31.3	46.9	24.2	36.2
	19	46.6	70.0	41.1	61.6	37.4	56.1	33.2	49.7	28.1	42.1	21.7	32.5
	20	42.1	63.1	37.1	55.6	33.8	50.7	29.9	44.9	25.3	38.0	19.6	29.4
	21	38.2	57.3	33.6	50.4	30.6	45.9	27.1	40.7	23.0	34.5	17.8	26.6
	22	34.8	52.2	30.6	45.9	27.9	41.9	24.7	37.1	20.9	31.4	16.2	24.3
	23	31.8	47.7	28.0	42.0	25.5	38.3	22.6	33.9	19.2	28.7	14.8	22.2
	24	29.2	43.8	25.7	38.6	23.5	35.2	20.8	31.2	17.6	26.4	13.6	20.4
	25	26.9	40.4	23.7	35.6	21.6	32.4	19.2	28.7	16.2	24.3	12.5	18.8
	26	24.9	37.4	21.9	32.9	20.0	30.0	17.7	26.6	15.0	22.5	11.6	17.4
	27			20.3	30.5	18.5	27.8	16.4	24.6	13.9	20.9	10.7	16.1
	28					17.2	25.8	15.3	22.9	12.9	19.4	10.0	15.0
	29									12.1	18.1	9.31	14.0

Properties													
M_n/Ω_b	ϕ_bM_n	kip-ft	23.4	35.2	19.2	28.8	16.7	25.1	13.9	20.9	10.8	16.3	7.50	11.3
$P_e(KL)^2/10^4$		kip-in.^{2}	553		487		444		393		333		258	

ASD	LRFD	
Ω_c = 2.00	ϕ_c = 0.75	Note: Heavy line indicates *Kl/r* equal to or greater than 200.

4

COMPOSITE
HSS4

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

$F_y = 46$ ksi
$f'_c = 4$ ksi

Shape		HSS4×4×											
		1/2		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		21.5		17.2		14.8		12.2		9.40		6.45	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	154	231	128	193	114	171	98.7	148	82.1	123	64.8	97.2
	1	153	230	128	192	113	170	98.2	147	81.7	123	64.5	96.7
	2	151	226	126	189	112	168	96.9	145	80.6	121	63.6	95.4
	3	147	221	123	184	109	164	94.7	142	78.9	118	62.2	93.3
	4	142	213	119	178	106	159	91.8	138	76.4	115	60.3	90.4
	5	135	203	114	171	101	152	88.1	132	73.4	110	57.9	86.8
	6	128	192	108	162	96.4	145	83.8	126	69.9	105	55.1	82.6
	7	120	180	101	152	90.7	136	79.0	118	66.0	99.0	51.9	77.9
	8	111	166	94.3	141	84.6	127	73.8	111	61.8	92.6	48.5	72.8
	9	101	152	86.9	130	78.1	117	68.3	102	57.3	85.9	44.9	67.4
	10	92.0	138	79.3	119	71.5	107	62.7	94.0	52.6	78.9	41.2	61.9
	11	82.6	124	71.7	108	64.8	97.2	57.0	85.4	47.9	71.9	37.5	56.3
	12	73.3	110	64.2	96.2	58.2	87.3	51.3	76.9	43.3	64.9	33.8	50.7
	13	64.5	96.7	56.9	85.3	51.8	77.7	45.8	68.7	38.7	58.1	30.2	45.3
	14	56.0	83.9	49.9	74.9	45.7	68.5	40.5	60.8	34.3	51.5	26.7	40.1
	15	48.7	73.1	43.5	65.2	39.8	59.8	35.4	53.2	30.1	45.2	23.4	35.1
	16	42.8	64.3	38.2	57.3	35.0	52.5	31.1	46.7	26.5	39.7	20.6	30.9
	17	37.9	56.9	33.9	50.8	31.0	46.5	27.6	41.4	23.5	35.2	18.2	27.3
	18	33.8	50.8	30.2	45.3	27.7	41.5	24.6	36.9	20.9	31.4	16.3	24.4
	19	30.4	45.6	27.1	40.7	24.8	37.2	22.1	33.1	18.8	28.2	14.6	21.9
	20	27.4	41.1	24.5	36.7	22.4	33.6	19.9	29.9	16.9	25.4	13.2	19.8
	21	24.9	37.3	22.2	33.3	20.3	30.5	18.1	27.1	15.4	23.1	11.9	17.9
	22	22.7	34.0	20.2	30.3	18.5	27.8	16.5	24.7	14.0	21.0	10.9	16.3
	23	20.7	31.1	18.5	27.7	16.9	25.4	15.1	22.6	12.8	19.2	10.0	14.9
	24			17.0	25.5	15.6	23.3	13.8	20.8	11.8	17.6	9.15	13.7
	25							12.8	19.1	10.8	16.3	8.43	12.6
	26											7.79	11.7

Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	17.7	26.6	14.7	22.1	12.8	19.3	10.8	16.2	8.42	12.7	5.87	8.82
$P_e(KL)^2/10^4$	kip-in.^{2}	360		321		294		262		223		173	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.											
$\Omega_c = 2.00$	$\phi_c = 0.75$												

F_y = 46 ksi
f'_c = 4 ksi

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

4
COMPOSITE
HSS3½

Shape		HSS3½×3½×									
		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		14.6		12.7		10.5		8.13		5.60	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	107	161	95.2	143	82.4	124	68.3	102	53.5	80.2
	1	106	160	94.7	142	81.9	123	67.9	102	53.1	79.7
	2	104	157	92.9	139	80.4	121	66.7	100	52.2	78.3
	3	101	152	90.0	135	78.0	117	64.8	97.2	50.7	76.1
	4	96.6	145	86.2	129	74.8	112	62.2	93.3	48.7	73.0
	5	91.1	137	81.5	122	70.9	106	59.0	88.5	46.2	69.3
	6	84.9	127	76.1	114	66.3	99.5	55.3	83.0	43.3	65.0
	7	78.0	117	70.2	105	61.3	92.0	51.3	76.9	40.2	60.2
	8	70.8	106	63.9	95.9	56.0	84.0	47.0	70.4	36.8	55.2
	9	63.4	95.1	57.5	86.2	50.6	75.9	42.5	63.8	33.3	50.0
	10	56.1	84.1	51.1	76.6	45.1	67.7	38.0	57.0	29.8	44.7
	11	49.0	73.4	44.8	67.2	39.7	59.6	33.6	50.4	26.4	39.6
	12	42.2	63.3	38.8	58.2	34.6	51.9	29.4	44.1	23.1	34.6
	13	35.9	53.9	33.2	49.8	29.7	44.6	25.3	38.0	19.9	29.8
	14	31.0	46.5	28.6	42.9	25.6	38.4	21.8	32.8	17.2	25.7
	15	27.0	40.5	24.9	37.4	22.3	33.5	19.0	28.5	14.9	22.4
	16	23.7	35.6	21.9	32.9	19.6	29.4	16.7	25.1	13.1	19.7
	17	21.0	31.5	19.4	29.1	17.4	26.1	14.8	22.2	11.6	17.5
	18	18.8	28.1	17.3	26.0	15.5	23.2	13.2	19.8	10.4	15.6
	19	16.8	25.2	15.5	23.3	13.9	20.9	11.9	17.8	9.31	14.0
	20	15.2	22.8	14.0	21.0	12.5	18.8	10.7	16.1	8.41	12.6
	21	13.8	20.7	12.7	19.1	11.4	17.1	9.71	14.6	7.62	11.4
	22					10.4	15.6	8.85	13.3	6.95	10.4

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	10.8	16.2	9.50	14.3	8.03	12.1	6.33	9.51	4.44	6.67
$P_e(KL)^2/10^4$	kip-in.^{2}	200		184		164		141		110	
ASD	**LRFD**	Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
Ω_c = 2.00	ϕ_c = 0.75										

4

COMPOSITE
HSS3

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS3×3×									
		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		12.1		10.5		8.78		6.85		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length (*KL*) (ft)	0	86.7	130	77.3	116	66.9	100	55.4	83.0	43.0	64.5
	1	85.9	129	76.7	115	66.4	99.6	54.9	82.4	42.6	64.0
	2	83.6	125	74.7	112	64.7	97.1	53.6	80.4	41.7	62.5
	3	79.9	120	71.5	107	62.1	93.1	51.5	77.2	40.0	60.1
	4	74.9	112	67.2	101	58.5	87.8	48.6	73.0	37.9	56.8
	5	69.0	103	62.1	93.2	54.3	81.4	45.2	67.9	35.3	52.9
	6	62.3	93.5	56.4	84.6	49.5	74.2	41.4	62.1	32.4	48.6
	7	55.3	83.0	50.4	75.5	44.4	66.6	37.3	55.9	29.2	43.8
	8	48.2	72.3	44.2	66.2	39.1	58.7	33.0	49.5	26.0	38.9
	9	41.3	61.9	38.1	57.1	33.9	50.9	28.8	43.2	22.7	34.1
	10	34.7	52.0	32.2	48.3	28.9	43.4	24.7	37.0	19.5	29.3
	11	28.7	43.0	26.8	40.2	24.2	36.3	20.8	31.2	16.5	24.8
	12	24.1	36.2	22.5	33.8	20.3	30.5	17.5	26.2	13.9	20.9
	13	20.5	30.8	19.2	28.8	17.3	26.0	14.9	22.3	11.8	17.8
	14	17.7	26.6	16.5	24.8	14.9	22.4	12.8	19.3	10.2	15.3
	15	15.4	23.1	14.4	21.6	13.0	19.5	11.2	16.8	8.90	13.3
	16	13.6	20.3	12.7	19.0	11.4	17.2	9.83	14.7	7.82	11.7
	17	12.0	18.0	11.2	16.8	10.1	15.2	8.71	13.1	6.93	10.4
	18			10.0	15.0	9.04	13.6	7.77	11.7	6.18	9.27
	19							6.97	10.5	5.55	8.32

Properties

		3/8 ASD	3/8 LRFD	5/16 ASD	5/16 LRFD	1/4 ASD	1/4 LRFD	3/16 ASD	3/16 LRFD	1/8 ASD	1/8 LRFD
M_n/Ω_b	$\phi_b M_n$ kip-ft	7.46	11.2	6.66	10.0	5.69	8.55	4.53	6.81	3.21	4.82
$P_e(KL)^2/10^4$	kip-in.^{2}	114		106		96.2		82.7		65.8	

ASD	LRFD	
$\Omega_c = 2.00$	$\phi_c = 0.75$	Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 4 ksi

Table 4–15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

4
COMPOSITE
HSS2½–HSS2¼

Shape		HSS2½×2½×								HSS2¼×2¼×	
		5/16		1/4		3/16		1/8		1/4	
t_{design}, in.		0.291		0.233		0.174		0.116		0.233	
Steel Wt/ft		8.40		7.08		5.57		3.90		6.23	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	60.3	90.4	52.3	78.5	43.3	64.9	33.4	50.1	45.4	68.0
	1	59.5	89.2	51.7	77.5	42.8	64.1	33.0	49.5	44.6	67.0
	2	57.1	85.7	49.8	74.7	41.3	61.9	31.9	47.8	42.6	63.9
	3	53.5	80.2	46.8	70.1	38.9	58.3	30.1	45.2	39.3	59.0
	4	48.8	73.2	42.8	64.3	35.8	53.7	27.8	41.7	35.2	52.8
	5	43.3	65.0	38.3	57.4	32.1	48.2	25.1	37.6	30.5	45.8
	6	37.4	56.2	33.4	50.1	28.2	42.3	22.1	33.2	25.7	38.5
	7	31.5	47.3	28.4	42.6	24.2	36.3	19.1	28.6	20.9	31.4
	8	25.9	38.8	23.5	35.3	20.2	30.3	16.1	24.1	16.5	24.7
	9	20.7	31.0	19.0	28.5	16.5	24.8	13.2	19.8	13.0	19.5
	10	16.7	25.1	15.4	23.1	13.4	20.1	10.7	16.1	10.5	15.8
	11	13.8	20.8	12.7	19.1	11.1	16.6	8.86	13.3	8.70	13.1
	12	11.6	17.4	10.7	16.0	9.29	13.9	7.45	11.2	7.31	11.0
	13	9.91	14.9	9.10	13.6	7.91	11.9	6.35	9.5	6.23	9.35
	14	8.54	12.8	7.84	11.8	6.82	10.2	5.47	8.21		
	15			6.83	10.2	5.94	8.91	4.77	7.15		
	16							4.19	6.28		

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	4.32	6.49	3.75	5.64	3.03	4.55	2.17	3.27	2.93	4.41
$P_e(KL)^2/10^4$	kip-in.^{2}	55.0		50.5		43.9		35.3		34.6	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
Ω_c = 2.00	ϕ_c = 0.75										

4

COMPOSITE
HSS2¼–HSS2

Table 4-15 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 4 ksi

Shape		HSS2¼×2¼×				HSS2×2×					
		3/16		1/8		1/4		3/16		1/8	
t_{design}, in.		0.174		0.116		0.233		0.174		0.116	
Steel Wt/ft		4.94		3.47		5.38		4.30		3.04	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	37.5	56.3	28.9	43.3	38.6	57.9	32.0	48.0	24.6	36.9
	1	37.0	55.5	28.5	42.7	37.8	56.7	31.4	47.1	24.2	36.2
	2	35.4	53.1	27.3	40.9	35.5	53.3	29.7	44.5	22.9	34.3
	3	32.8	49.2	25.4	38.1	32.0	48.1	26.9	40.4	20.9	31.3
	4	29.6	44.4	23.0	34.5	27.7	41.6	23.5	35.3	18.4	27.6
	5	25.9	38.8	20.2	30.4	23.0	34.5	19.8	29.7	15.6	23.4
	6	22.0	32.9	17.3	26.0	18.3	27.5	16.0	24.0	12.8	19.2
	7	18.1	27.1	14.4	21.6	14.0	21.0	12.5	18.7	10.1	15.2
	8	14.5	21.7	11.6	17.5	10.7	16.1	9.54	14.3	7.77	11.7
	9	11.4	17.1	9.21	13.8	8.47	12.7	7.54	11.3	6.14	9.21
	10	9.25	13.9	7.46	11.2	6.86	10.3	6.11	9.16	4.97	7.46
	11	7.65	11.5	6.17	9.25	5.67	8.50	5.05	7.57	4.11	6.16
	12	6.42	9.64	5.18	7.77			4.24	6.36	3.45	5.18
	13	5.47	8.21	4.42	6.62						
	14			3.81	5.71						

Properties											
M_n/Ω_b	$\phi_b M_n$	kip-ft	2.39	3.60	1.73	2.60	2.21	3.33	1.83	2.75	1.34	2.02
$P_e(KL)^2/10^4$		kip-in.^{2}	30.4		24.5		22.5		20.0		16.3	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4–16
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

5

COMPOSITE
HSS16–HSS14

Shape		HSS16×16×						HSS14×14×					
		1/2		3/8		5/16		5/8		1/2		3/8	
t_{design}, in.		0.465		0.349		0.291		0.581		0.465		0.349	
Steel Wt/ft		103		78.4		65.8		110		89.6		68.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	1130	1700	993	1490	921	1380	1050	1570	929	1390	807	1210
	6	1120	1680	982	1470	910	1360	1030	1550	916	1370	795	1190
	7	1120	1680	978	1470	906	1360	1030	1540	912	1370	791	1190
	8	1110	1670	973	1460	902	1350	1020	1530	906	1360	787	1180
	9	1110	1660	968	1450	897	1350	1020	1520	900	1350	781	1170
	10	1100	1650	962	1440	891	1340	1010	1510	894	1340	775	1160
	11	1090	1640	956	1430	885	1330	1000	1500	887	1330	769	1150
	12	1090	1630	949	1420	879	1320	991	1490	879	1320	762	1140
	13	1080	1620	942	1410	872	1310	982	1470	870	1310	754	1130
	14	1070	1600	934	1400	864	1300	972	1460	861	1290	746	1120
	15	1060	1590	926	1390	856	1280	961	1440	852	1280	738	1110
	16	1050	1580	917	1370	848	1270	950	1430	842	1260	729	1090
	17	1040	1560	907	1360	839	1260	938	1410	831	1250	719	1080
	18	1030	1540	897	1350	830	1240	926	1390	820	1230	709	1060
	19	1020	1530	887	1330	820	1230	913	1370	809	1210	699	1050
	20	1010	1510	876	1310	810	1210	900	1350	796	1190	688	1030
	21	994	1490	865	1300	799	1200	886	1330	784	1180	677	1020
	22	981	1470	854	1280	788	1180	872	1310	771	1160	665	998
	23	968	1450	842	1260	777	1170	857	1290	758	1140	654	980
	24	955	1430	830	1240	765	1150	842	1260	744	1120	641	962
	25	941	1410	817	1230	753	1130	826	1240	731	1100	629	944
	26	927	1390	804	1210	741	1110	811	1220	716	1070	616	925
	27	912	1370	791	1190	728	1090	795	1190	702	1050	603	905
	28	897	1350	778	1170	716	1070	778	1170	687	1030	590	886
	29	882	1320	764	1150	703	1050	762	1140	672	1010	577	866
	30	867	1300	750	1130	689	1030	745	1120	657	986	564	846
	32	835	1250	722	1080	663	994	711	1070	627	940	537	805
	34	803	1200	693	1040	635	953	676	1010	596	894	509	764
	36	770	1160	663	995	607	911	641	962	564	847	481	722
	38	737	1110	633	950	579	868	606	909	533	800	454	681
	40	704	1060	603	905	551	826	571	857	502	753	427	640

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	376	566	289	435	243	366	347	521	285	428	219	329
$P_e(KL)^2/10^4$		kip-in.^{2}	45800		38400		34600		33400		29100		24500	
ASD	LRFD													
$\Omega_c = 2.00$	$\phi_c = 0.75$													

5

COMPOSITE
HSS14–HSS12

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS14×14×		HSS12×12×									
		5/16		5/8		1/2		3/8		5/16		1/4	
t_{design}, in.		0.291		0.581		0.465		0.349		0.291		0.233	
Steel Wt/ft		57.3		93.1		75.9		58.0		48.8		39.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	744	1120	839	1260	741	1110	638	957	585	878	532	797
	6	734	1100	824	1240	727	1090	626	939	574	861	521	781
	7	730	1090	818	1230	722	1080	622	933	570	855	517	776
	8	725	1090	812	1220	717	1070	617	925	565	848	513	769
	9	720	1080	805	1210	710	1070	611	917	560	840	508	762
	10	715	1070	797	1200	703	1060	605	908	554	831	502	754
	11	708	1060	788	1180	696	1040	598	898	548	822	496	745
	12	702	1050	779	1170	688	1030	591	887	541	812	490	735
	13	695	1040	769	1150	679	1020	583	875	534	801	483	725
	14	687	1030	758	1140	669	1000	575	862	526	789	476	714
	15	679	1020	747	1120	659	989	566	849	518	777	468	702
	16	670	1010	735	1100	649	973	557	835	509	764	460	690
	17	661	992	723	1080	638	957	547	821	500	750	452	677
	18	652	978	710	1060	627	940	537	806	491	736	443	664
	19	642	963	696	1040	615	922	527	790	481	721	434	650
	20	632	948	682	1020	603	904	516	774	471	706	424	636
	21	621	932	668	1000	590	885	505	757	460	691	414	622
	22	611	916	653	980	577	865	493	740	450	675	404	607
	23	599	899	638	957	564	846	482	723	439	658	394	591
	24	588	882	623	934	550	825	470	705	428	642	384	576
	25	576	864	607	911	536	805	458	687	417	625	373	560
	26	564	847	592	887	522	784	446	668	405	608	363	544
	27	552	828	576	863	508	763	433	650	394	590	352	528
	28	540	810	559	839	494	741	421	631	382	573	341	512
	29	527	791	543	815	480	720	408	612	370	556	331	496
	30	515	772	527	790	465	698	396	593	359	538	320	480
	32	489	734	494	741	437	655	370	556	335	503	298	447
	34	464	695	461	692	408	612	345	518	312	468	277	415
	36	438	657	429	644	379	569	321	481	289	434	256	383
	38	412	618	398	596	351	527	296	444	267	400	235	353
	40	386	580	367	550	324	486	273	409	245	368	215	323

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	185	278	250	376	206	309	159	239	134	202	109	164
$P_e(KL)^2/10^4$	kip-in.^{2}	21900		19600		17300		14500		13000		11300	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 46 ksi
f'_c = 5 ksi

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

5
COMPOSITE HSS10

Shape		HSS10×10×											
		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		76.1		62.3		47.8		40.3		32.6		24.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	649	973	570	854	486	729	443	665	399	599	353	530
	6	631	947	554	832	473	710	431	647	388	582	343	515
	7	625	938	549	824	469	703	427	640	384	576	339	509
	8	618	927	543	815	464	695	422	633	380	569	335	503
	9	610	916	536	804	458	687	417	625	374	562	330	496
	10	602	903	529	793	451	677	411	616	369	553	325	488
	11	592	888	520	781	444	666	404	606	363	544	320	479
	12	582	873	512	767	437	655	397	595	356	534	314	470
	13	571	857	502	753	428	643	389	584	349	524	307	461
	14	560	840	492	738	420	630	381	572	342	513	300	451
	15	548	822	482	723	411	616	373	560	334	501	293	440
	16	535	803	471	706	401	602	364	547	326	489	286	429
	17	522	783	459	689	392	587	355	533	318	477	278	417
	18	508	763	448	671	382	572	346	519	309	464	270	405
	19	494	742	435	653	371	557	336	504	300	450	262	393
	20	480	720	423	634	360	541	326	489	291	437	254	380
	21	465	698	410	615	350	524	316	474	282	423	245	368
	22	451	676	397	596	338	508	306	459	272	409	236	355
	23	436	653	384	576	327	491	296	443	263	394	228	342
	24	420	631	371	557	316	474	285	428	253	380	219	329
	25	405	608	358	537	305	457	275	412	244	365	210	316
	26	390	585	344	517	293	440	264	396	234	351	202	302
	27	375	562	331	497	282	423	254	380	224	337	193	289
	28	359	539	318	477	270	406	243	365	215	322	184	277
	29	344	517	305	457	259	389	233	349	205	308	176	264
	30	329	494	292	437	248	372	223	334	196	294	167	251
	32	300	450	266	399	226	339	202	304	178	267	151	227
	34	272	407	241	361	205	307	183	274	160	240	135	203
	36	244	366	217	325	184	276	164	246	143	215	121	181
	38	219	328	195	292	165	248	147	221	128	193	108	162
	40	198	296	176	263	149	223	133	199	116	174	97.7	147

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	168	252	139	210	108	163	92.0	138	75.0	113	57.0	85.6
$P_e(KL)^2/10^4$	kip-in.^{2}	10400		9230		7830		6980		6090		5130	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

5

COMPOSITE
HSS9

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS9×9×											
		5/8		1/2		3/8		5/16		1/4		3/16	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		67.6		55.5		42.7		36.0		29.2		22.2	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	560	840	490	735	417	625	379	568	339	509	299	448
	6	541	812	474	711	403	605	366	549	328	492	288	432
	7	535	802	469	703	398	598	362	542	324	486	284	426
	8	527	791	462	693	393	590	357	535	319	479	280	420
	9	519	779	455	683	387	580	351	527	314	471	275	413
	10	510	765	447	671	380	570	345	517	308	463	270	405
	11	500	750	439	658	373	560	338	507	302	453	264	397
	12	489	734	429	644	365	548	331	497	296	443	258	387
	13	478	717	420	630	357	536	323	485	289	433	252	378
	14	466	699	409	614	348	522	315	473	281	422	245	368
	15	454	680	399	598	339	509	307	460	274	410	238	357
	16	441	661	387	581	330	494	298	447	266	398	231	346
	17	427	641	376	564	320	480	289	434	257	386	223	335
	18	413	620	364	546	310	465	280	420	249	373	216	323
	19	399	599	352	528	299	449	270	406	240	360	208	312
	20	385	578	339	509	289	433	261	391	231	347	200	300
	21	371	556	327	490	278	417	251	376	222	334	192	288
	22	356	534	314	471	268	401	241	362	213	320	184	275
	23	341	512	301	452	257	385	231	347	204	307	175	263
	24	327	490	289	433	246	369	221	332	195	293	167	251
	25	312	468	276	414	235	353	211	317	186	280	159	239
	26	297	446	263	395	224	337	202	302	178	266	151	227
	27	283	425	251	376	214	321	192	288	169	253	144	215
	28	269	403	238	357	203	305	182	273	160	240	136	204
	29	255	382	226	339	193	289	173	259	152	227	128	192
	30	241	362	214	321	183	274	164	245	143	215	121	181
	32	214	322	191	286	163	245	146	218	127	190	107	160
	34	190	285	169	254	144	217	129	193	112	169	94.4	142
	36	169	254	151	226	129	193	115	173	100	150	84.2	126
	38	152	228	135	203	116	173	103	155	90.0	135	75.5	113
	40	137	206	122	183	104	157	93.2	140	81.2	122	68.2	102
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	133	200	111	167	86.8	130	73.8	111	60.2	90.5	45.9	69.0
$P_e(KL)^2/10^4$	kip-in.^{2}	7210		6420		5490		4900		4260		3590	
ASD	LRFD												
Ω_c = 2.00	ϕ_c = 0.75												

F_y = 46 ksi
f'_c = 5 ksi

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

5
COMPOSITE HSS8

Shape		HSS8×8×									
		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$	
t_{design}, in.		0.581		0.465		0.349		0.291		0.233	
Steel Wt/ft		59.1		48.7		37.6		31.8		25.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	475	713	415	623	352	527	318	477	284	426
	6	455	683	398	597	337	506	305	458	272	408
	7	448	672	392	588	332	498	300	451	268	401
	8	440	660	385	578	326	490	295	443	263	394
	9	431	647	378	567	320	480	289	434	258	386
	10	421	632	369	554	313	470	283	425	252	378
	11	411	616	360	541	306	458	276	414	246	368
	12	400	600	351	526	298	446	269	403	239	358
	13	388	582	341	511	289	434	261	392	232	348
	14	376	563	330	495	280	420	253	380	225	337
	15	363	544	319	478	271	406	245	367	217	325
	16	349	524	308	461	261	392	236	354	209	314
	17	336	504	296	444	252	377	227	341	201	301
	18	322	483	284	426	242	362	218	327	193	289
	19	308	462	272	408	232	347	209	313	184	277
	20	294	441	260	390	221	332	199	299	176	264
	21	280	420	248	371	211	317	190	285	168	251
	22	266	398	235	353	201	301	181	271	159	239
	23	252	377	223	335	191	286	172	257	151	226
	24	238	357	211	317	181	271	162	244	143	214
	25	224	336	199	299	171	256	153	230	135	202
	26	211	316	188	282	161	241	145	217	127	190
	27	198	297	177	265	151	227	136	204	119	178
	28	185	278	166	248	142	213	127	191	111	167
	29	173	259	155	232	133	199	119	179	104	156
	30	161	242	144	217	124	186	111	167	96.9	145
	32	142	213	127	190	109	163	97.7	147	85.2	128
	34	126	188	112	169	96.5	145	86.6	130	75.4	113
	36	112	168	100	150	86.1	129	77.2	116	67.3	101
	38	100	151	90.0	135	77.3	116	69.3	104	60.4	90.6
	40	90.7	136	81.2	122	69.7	105	62.6	93.8	54.5	81.8
Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	103	154	86.0	129	67.6	102	57.6	86.6	47.1	70.9
$P_e(KL)^2/10^4$	kip-in.^{2}	4770		4260		3660		3280		2870	
ASD	LRFD										
$\Omega_c = 2.00$	$\phi_c = 0.75$										

5

COMPOSITE HSS8–HSS7

Table 4-16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS8×8×		HSS7×7×							
		$^3/_{16}$		$^5/_8$		$^1/_2$		$^3/_8$		$^5/_{16}$	
t_{design}, in.		0.174		0.581		0.465		0.349		0.291	
Steel Wt/ft		19.6		50.6		41.9		32.5		27.5	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	248	372	395	592	345	517	291	436	262	393
	6	237	355	373	559	326	489	275	413	248	372
	7	233	350	365	548	319	479	270	404	243	365
	8	229	343	357	535	312	468	264	395	238	357
	9	224	336	347	520	304	456	257	385	232	348
	10	219	328	337	505	295	443	249	374	225	338
	11	213	320	325	488	286	428	242	362	218	327
	12	207	311	314	471	276	413	233	350	211	316
	13	201	301	301	452	265	398	225	337	203	304
	14	194	291	289	433	254	381	216	323	195	292
	15	187	281	276	413	243	364	206	309	186	279
	16	180	270	262	393	232	347	197	295	178	267
	17	173	259	249	373	220	330	187	281	169	254
	18	166	248	235	353	208	312	177	266	160	241
	19	158	237	222	333	197	295	168	252	152	227
	20	151	226	208	312	185	278	158	237	143	214
	21	143	215	195	293	174	261	148	223	134	202
	22	136	203	182	273	162	244	139	209	126	189
	23	128	192	169	254	151	227	130	195	118	176
	24	121	181	157	236	141	211	121	181	110	164
	25	114	171	145	218	130	195	112	168	102	152
	26	107	160	134	201	120	181	104	155	93.9	141
	27	99.9	150	124	187	112	167	96.1	144	87.1	131
	28	93.1	140	116	173	104	156	89.3	134	81.0	121
	29	86.8	130	108	162	96.8	145	83.3	125	75.5	113
	30	81.1	122	101	151	90.4	136	77.8	117	70.6	106
	32	71.3	107	88.5	133	79.5	119	68.4	103	62.0	93.0
	34	63.2	94.7	78.4	118	70.4	106	60.6	90.9	54.9	82.4
	36	56.3	84.5	69.9	105	62.8	94.2	54.0	81.1	49.0	73.5
	38	50.6	75.8	62.8	94.2	56.4	84.6	48.5	72.8	44.0	66.0
	40	45.6	68.4	56.7	85.0	50.9	76.3	43.8	65.7	39.7	59.5
Properties											
M_n/Ω_b | ϕ_bM_n	kip-ft	36.0	54.1	75.9	114	64.1	96.4	50.7	76.2	43.4	65.2
$P_e(KL)^2/10^4$	kip-in.^{2}	2400		2980		2680		2300		2090	
ASD	LRFD										
Ω_c = 2.00	ϕ_c = 0.75										

F_y = 46 ksi
f'_c = 5 ksi

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

5
COMPOSITE
HSS7–HSS6

Shape		HSS7×7×						HSS6×6×			
		1/4		3/16		1/8		5/8		1/2	
t_{design}, in.		0.233		0.174		0.116		0.581		0.465	
Steel Wt/ft		22.4		17.1		11.6		42.1		35.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	232	349	201	302	170	255	319	478	278	417
	6	220	330	190	285	160	240	294	441	257	386
	7	216	323	186	279	156	235	286	429	250	375
	8	211	316	182	273	152	229	276	414	242	364
	9	205	308	177	266	148	222	266	399	234	350
	10	199	299	172	258	143	215	255	382	224	336
	11	193	290	166	249	138	207	243	365	214	322
	12	186	279	160	240	133	199	231	346	204	306
	13	179	269	154	231	127	191	218	328	193	290
	14	172	258	147	221	122	182	206	308	182	274
	15	165	247	141	211	116	174	193	289	171	257
	16	157	235	134	201	110	165	180	270	160	240
	17	149	224	127	191	104	156	167	250	149	224
	18	141	212	120	180	97.7	147	154	232	138	208
	19	133	200	113	170	91.8	138	142	213	128	192
	20	126	189	106	160	85.8	129	130	195	118	176
	21	118	177	99.7	150	80.0	120	119	178	107	161
	22	110	166	93.1	140	74.4	112	108	162	97.9	147
	23	103	155	86.7	130	68.9	103	98.8	148	89.6	134
	24	95.9	144	80.4	121	63.4	95.1	90.8	136	82.3	123
	25	88.8	133	74.2	111	58.5	87.7	83.6	125	75.8	114
	26	82.1	123	68.6	103	54.0	81.1	77.3	116	70.1	105
	27	76.2	114	63.6	95.5	50.1	75.2	71.7	108	65.0	97.5
	28	70.8	106	59.2	88.8	46.6	69.9	66.7	100	60.5	90.7
	29	66.0	99.0	55.2	82.7	43.4	65.2	62.2	93.2	56.4	84.5
	30	61.7	92.6	51.5	77.3	40.6	60.9	58.1	87.1	52.7	79.0
	32	54.2	81.3	45.3	68.0	35.7	53.5	51.1	76.6	46.3	69.4
	34	48.0	72.1	40.1	60.2	31.6	47.4	45.2	67.8	41.0	61.5
	36	42.8	64.3	35.8	53.7	28.2	42.3	40.3	60.5	36.6	54.9
	38	38.5	57.7	32.1	48.2	25.3	38.0				
	40	34.7	52.1	29.0	43.5	22.8	34.3				
Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	35.6	53.6	27.3	41.0	18.7	28.0	53.2	80.0	45.4	68.3
$P_e(KL)^2/10^4$	kip-in.^{2}	1830		1530		1200		1710		1560	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

5

COMPOSITE
HSS6

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS6×6×									
		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		27.4		23.3		19.0		14.5		9.85	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	234	351	210	315	185	278	159	239	133	199
	6	217	325	195	292	172	258	148	221	122	183
	7	211	316	190	285	168	251	144	215	119	178
	8	204	307	184	276	162	244	139	209	115	172
	9	197	296	178	266	157	235	134	201	110	166
	10	190	284	171	256	151	226	129	193	106	159
	11	181	272	164	245	144	217	123	185	101	151
	12	173	259	156	234	138	207	117	176	95.8	144
	13	164	246	148	222	131	196	111	167	90.6	136
	14	155	233	140	210	124	186	105	158	85.2	128
	15	146	219	132	198	117	175	98.8	148	79.8	120
	16	137	205	124	185	109	164	92.5	139	74.4	112
	17	128	192	115	173	102	153	86.2	129	69.0	104
	18	119	178	107	161	95.0	142	80.0	120	63.8	95.7
	19	110	165	99.5	149	88.0	132	74.0	111	58.7	88.0
	20	101	152	91.8	138	81.2	122	68.1	102	53.7	80.6
	21	92.9	139	84.3	126	74.6	112	62.4	93.6	48.9	73.3
	22	84.8	127	77.0	115	68.1	102	56.9	85.3	44.5	66.8
	23	77.6	116	70.4	106	62.3	93.5	52.0	78.0	40.7	61.1
	24	71.2	107	64.7	97.0	57.2	85.9	47.8	71.7	37.4	56.1
	25	65.7	98.5	59.6	89.4	52.8	79.1	44.0	66.1	34.5	51.7
	26	60.7	91.0	55.1	82.7	48.8	73.2	40.7	61.1	31.9	47.8
	27	56.3	84.4	51.1	76.6	45.2	67.8	37.8	56.6	29.6	44.3
	28	52.3	78.5	47.5	71.3	42.1	63.1	35.1	52.7	27.5	41.2
	29	48.8	73.2	44.3	66.4	39.2	58.8	32.7	49.1	25.6	38.4
	30	45.6	68.4	41.4	62.1	36.6	55.0	30.6	45.9	23.9	35.9
	32	40.1	60.1	36.4	54.6	32.2	48.3	26.9	40.3	21.0	31.6
	34	35.5	53.2	32.2	48.3	28.5	42.8	23.8	35.7	18.6	28.0
	36	31.7	47.5	28.7	43.1	25.4	38.2	21.2	31.9	16.6	24.9
	38	28.4	42.6	25.8	38.7	22.8	34.2	19.1	28.6	14.9	22.4
Properties											
M_n/Ω_b | $\phi_b M_n$	kip-ft	36.3	54.6	31.2	46.9	25.7	38.7	19.8	29.8	13.6	20.4
$P_e(KL)^2/10^4$	kip-in.^{2}	1350		1220		1080		904		707	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 46 ksi
f'_c = 5 ksi

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

5

COMPOSITE
HSS5$^1/_2$–HSS5

Shape		HSS5$^1/_2$×5$^1/_2$×										HSS5×5×	
		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$		$^1/_2$	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116		0.465	
Steel Wt/ft		24.9		21.2		17.3		13.2		9.00		28.3	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	207	310	186	279	163	245	140	210	116	173	216	324
	1	206	310	185	278	163	245	139	209	115	173	215	323
	2	205	307	184	276	162	243	138	208	114	172	213	320
	3	202	304	182	272	160	240	137	205	113	169	210	315
	4	199	298	178	268	157	236	134	201	111	166	205	308
	5	194	292	175	262	154	231	131	197	108	162	199	299
	6	189	284	170	255	150	224	128	192	105	158	193	289
	7	183	275	164	247	145	217	124	186	102	152	185	277
	8	176	264	158	238	140	210	119	179	97.6	146	176	264
	9	169	253	152	228	134	201	114	171	93.3	140	167	250
	10	161	242	145	217	128	192	109	163	88.7	133	157	235
	11	153	229	138	206	121	182	103	155	83.9	126	147	220
	12	144	216	130	195	115	172	97.5	146	78.9	118	136	204
	13	135	203	122	183	108	162	91.6	137	73.9	111	126	189
	14	127	190	114	172	101	152	85.6	128	68.8	103	115	173
	15	118	177	106	160	94.1	141	79.6	119	63.7	95.5	105	158
	16	109	163	98.6	148	87.2	131	73.7	111	58.7	88.0	95.3	143
	17	100	150	90.8	136	80.4	121	67.8	102	53.8	80.6	85.7	129
	18	91.9	138	83.3	125	73.8	111	62.1	93.2	49.0	73.5	76.5	115
	19	83.7	126	76.0	114	67.4	101	56.7	85.0	44.4	66.5	68.7	103
	20	75.8	114	68.9	103	61.1	91.7	51.3	76.9	40.0	60.1	62.0	93.0
	21	68.7	103	62.5	93.7	55.4	83.2	46.5	69.8	36.3	54.5	56.2	84.3
	22	62.6	93.9	56.9	85.4	50.5	75.8	42.4	63.6	33.1	49.6	51.2	76.9
	23	57.3	85.9	52.1	78.1	46.2	69.3	38.8	58.2	30.3	45.4	46.9	70.3
	24	52.6	78.9	47.8	71.8	42.4	63.7	35.6	53.4	27.8	41.7	43.1	64.6
	25	48.5	72.7	44.1	66.1	39.1	58.7	32.8	49.2	25.6	38.4	39.7	59.5
	26	44.8	67.3	40.8	61.1	36.2	54.3	30.3	45.5	23.7	35.5	36.7	55.0
	27	41.6	62.4	37.8	56.7	33.5	50.3	28.1	42.2	22.0	33.0	34.0	51.0
	28	38.7	58.0	35.1	52.7	31.2	46.8	26.2	39.2	20.4	30.6	31.6	47.4
	29	36.0	54.1	32.8	49.1	29.1	43.6	24.4	36.6	19.0	28.6	29.5	44.2
	30	33.7	50.5	30.6	45.9	27.2	40.7	22.8	34.2	17.8	26.7	27.6	41.3
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	30.0	45.1	25.9	38.9	21.4	32.2	16.5	24.8	11.4	17.1	30.0	45.0
$P_e(KL)^2/10^4$	kip-in.^{2}	995		904		804		674		526		815	
ASD	LRFD												
Ω_c = 2.00	ϕ_c = 0.75												

5

COMPOSITE
HSS5

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

$F_y = 46$ ksi
$f'_c = 5$ ksi

Shape		HSS5×5×									
		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		22.3		19.0		15.6		12.0		8.15	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	181	272	162	244	143	214	121	182	99.6	149
	1	181	271	162	243	142	213	121	182	99.3	149
	2	179	269	160	241	141	211	120	180	98.4	148
	3	176	264	158	237	139	208	118	177	96.8	145
	4	173	259	155	232	136	204	116	174	94.7	142
	5	168	252	151	226	132	198	113	169	92.0	138
	6	162	243	146	219	128	192	109	163	88.9	133
	7	156	234	140	210	123	185	105	157	85.3	128
	8	149	223	134	201	118	177	100	150	81.4	122
	9	141	212	127	191	112	168	95.2	143	77.1	116
	10	133	200	120	180	106	159	89.9	135	72.6	109
	11	125	188	113	169	99.4	149	84.5	127	68.0	102
	12	117	175	105	158	92.9	139	78.8	118	63.2	94.8
	13	108	162	97.6	146	86.2	129	73.1	110	58.4	87.6
	14	99.4	149	90.0	135	79.5	119	67.4	101	53.6	80.5
	15	91.0	136	82.5	124	73.0	109	61.8	92.7	49.0	73.4
	16	82.7	124	75.1	113	66.5	99.8	56.3	84.5	44.4	66.6
	17	74.8	112	68.0	102	60.3	90.5	51.0	76.5	40.0	60.0
	18	67.0	101	61.1	91.6	54.3	81.4	45.9	68.8	35.7	53.6
	19	60.2	90.3	54.8	82.2	48.7	73.1	41.2	61.7	32.1	48.1
	20	54.3	81.5	49.5	74.2	44.0	65.9	37.1	55.7	29.0	43.4
	21	49.3	73.9	44.9	67.3	39.9	59.8	33.7	50.5	26.3	39.4
	22	44.9	67.3	40.9	61.3	36.3	54.5	30.7	46.1	23.9	35.9
	23	41.1	61.6	37.4	56.1	33.2	49.9	28.1	42.1	21.9	32.8
	24	37.7	56.6	34.4	51.5	30.5	45.8	25.8	38.7	20.1	30.2
	25	34.8	52.1	31.7	47.5	28.1	42.2	23.8	35.7	18.5	27.8
	26	32.1	48.2	29.3	43.9	26.0	39.0	22.0	33.0	17.1	25.7
	27	29.8	44.7	27.2	40.7	24.1	36.2	20.4	30.6	15.9	23.8
	28	27.7	41.6	25.2	37.9	22.4	33.6	19.0	28.4	14.8	22.2
	29	25.8	38.7	23.5	35.3	20.9	31.4	17.7	26.5	13.8	20.7
	30	24.1	36.2	22.0	33.0	19.5	29.3	16.5	24.8	12.9	19.3
Properties											
M_n/Ω_b | $\phi_b M_n$	kip-ft	24.3	36.5	21.0	31.6	17.5	26.2	13.5	20.3	9.33	14.0
$P_e(KL)^2/10^4$	kip-in.^{2}	713		649		577		488		380	
ASD	LRFD										
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 46 ksi
f'_c = 5 ksi

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

5
COMPOSITE
HSS4$^1/_2$

Shape		HSS4$^1/_2$×4$^1/_2$×											
		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		24.9		19.7		16.9		13.9		10.7		7.30	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	186	280	157	235	140	210	123	184	104	156	84.7	127
	1	186	279	156	234	140	209	122	183	104	155	84.3	127
	2	184	275	154	231	138	207	121	181	103	154	83.4	125
	3	180	270	151	227	135	203	119	178	101	151	81.8	123
	4	175	262	147	221	132	198	116	174	98.1	147	79.6	119
	5	169	253	142	213	128	191	112	168	94.9	142	76.9	115
	6	161	242	136	205	122	184	107	161	91.2	137	73.7	111
	7	153	230	130	195	117	175	102	154	86.9	130	70.2	105
	8	144	216	123	184	110	165	96.9	145	82.3	123	66.2	99.4
	9	135	202	115	172	103	155	91.0	137	77.3	116	62.1	93.1
	10	125	187	107	160	96.3	144	84.9	127	72.1	108	57.7	86.5
	11	114	172	98.5	148	89.0	134	78.5	118	66.7	100	53.2	79.8
	12	104	156	90.2	135	81.7	122	72.1	108	61.3	92.0	48.7	73.1
	13	94.3	141	81.9	123	74.3	112	65.8	98.7	55.9	83.9	44.3	66.4
	14	84.6	127	73.9	111	67.2	101	59.5	89.3	50.7	76.0	39.9	59.9
	15	75.2	113	66.1	99.2	60.3	90.4	53.5	80.2	45.6	68.3	35.7	53.6
	16	66.3	99.4	58.6	87.9	53.6	80.4	47.7	71.5	40.7	61.0	31.6	47.5
	17	58.7	88.1	51.9	77.9	47.5	71.2	42.2	63.4	36.0	54.0	28.0	42.0
	18	52.4	78.5	46.3	69.4	42.3	63.5	37.7	56.5	32.1	48.2	25.0	37.5
	19	47.0	70.5	41.6	62.3	38.0	57.0	33.8	50.7	28.8	43.2	22.4	33.7
	20	42.4	63.6	37.5	56.3	34.3	51.4	30.5	45.8	26.0	39.0	20.3	30.4
	21	38.5	57.7	34.0	51.0	31.1	46.6	27.7	41.5	23.6	35.4	18.4	27.6
	22	35.1	52.6	31.0	46.5	28.3	42.5	25.2	37.8	21.5	32.2	16.7	25.1
	23	32.1	48.1	28.4	42.5	25.9	38.9	23.1	34.6	19.7	29.5	15.3	23.0
	24	29.5	44.2	26.0	39.1	23.8	35.7	21.2	31.8	18.1	27.1	14.1	21.1
	25	27.1	40.7	24.0	36.0	21.9	32.9	19.5	29.3	16.6	25.0	13.0	19.4
	26	25.1	37.6	22.2	33.3	20.3	30.4	18.1	27.1	15.4	23.1	12.0	18.0
	27			20.6	30.9	18.8	28.2	16.7	25.1	14.3	21.4	11.1	16.7
	28					17.5	26.2	15.6	23.4	13.3	19.9	10.3	15.5
	29									12.4	18.6	9.63	14.4

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	23.4	35.2	19.2	28.8	16.7	25.1	13.9	20.9	10.8	16.3	7.50	11.3
$P_e(KL)^2/10^4$	kip-in.^{2}	557		492		451		400		341		266	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.											
$\Omega_c = 2.00$	$\phi_c = 0.75$												

5

COMPOSITE
HSS4

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS4×4×											
		$^1/_2$		$^3/_8$		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.465		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		21.5		17.2		14.8		12.2		9.40		6.45	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	158	237	133	199	119	178	104	156	87.7	132	70.8	106
	1	157	236	132	198	118	178	103	155	87.3	131	70.5	106
	2	155	232	130	196	117	175	102	153	86.1	129	69.5	104
	3	151	226	127	191	114	171	99.7	149	84.1	126	67.8	102
	4	145	218	123	184	110	165	96.4	145	81.5	122	65.6	98.4
	5	139	208	118	176	105	158	92.4	139	78.1	117	62.8	94.2
	6	131	196	111	167	100	150	87.8	132	74.2	111	59.6	89.4
	7	122	183	104	157	94.0	141	82.6	124	69.9	105	56.0	84.0
	8	113	169	96.9	145	87.4	131	76.9	115	65.2	97.8	52.1	78.2
	9	103	155	89.1	134	80.6	121	71.0	107	60.2	90.4	48.1	72.1
	10	93.4	140	81.1	122	73.5	110	64.9	97.4	55.2	82.7	43.9	65.8
	11	83.7	125	73.1	110	66.5	99.7	58.8	88.3	50.0	75.1	39.7	59.6
	12	74.1	111	65.3	97.9	59.5	89.3	52.8	79.2	45.0	67.5	35.6	53.4
	13	65.0	97.5	57.7	86.6	52.8	79.2	46.9	70.4	40.0	60.1	31.6	47.3
	14	56.3	84.4	50.4	75.6	46.3	69.5	41.3	62.0	35.3	53.0	27.7	41.6
	15	49.0	73.6	43.9	65.9	40.4	60.5	36.0	54.1	30.8	46.3	24.2	36.2
	16	43.1	64.6	38.6	57.9	35.5	53.2	31.7	47.5	27.1	40.7	21.2	31.9
	17	38.2	57.3	34.2	51.3	31.4	47.1	28.1	42.1	24.0	36.0	18.8	28.2
	18	34.1	51.1	30.5	45.8	28.0	42.0	25.0	37.6	21.4	32.1	16.8	25.2
	19	30.6	45.8	27.4	41.1	25.2	37.7	22.5	33.7	19.2	28.8	15.1	22.6
	20	27.6	41.4	24.7	37.1	22.7	34.1	20.3	30.4	17.3	26.0	13.6	20.4
	21	25.0	37.5	22.4	33.6	20.6	30.9	18.4	27.6	15.7	23.6	12.3	18.5
	22	22.8	34.2	20.4	30.6	18.8	28.1	16.8	25.1	14.3	21.5	11.2	16.8
	23	20.9	31.3	18.7	28.0	17.2	25.7	15.3	23.0	13.1	19.7	10.3	15.4
	24			17.2	25.7	15.8	23.6	14.1	21.1	12.0	18.1	9.44	14.2
	25							13.0	19.5	11.1	16.7	8.70	13.0
	26											8.04	12.1

Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	17.7	26.6	14.7	22.1	12.8	19.3	10.8	16.2	8.42	12.7	5.87	8.82
$P_e(KL)^2/10^4$	kip-in.^{2}	363		324		298		266		228		179	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.											
Ω_c = 2.00	ϕ_c = 0.75												

F_y = 46 ksi
f'_c = 5 ksi

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

5
COMPOSITE
HSS3½

Shape		HSS3½×3½×									
		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		14.6		12.7		10.5		8.13		5.60	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	110	166	98.8	148	86.3	129	72.5	109	58.0	87.0
	1	110	165	98.2	147	85.7	129	72.1	108	57.6	86.4
	2	108	161	96.3	145	84.2	126	70.8	106	56.6	84.9
	3	104	156	93.3	140	81.6	122	68.6	103	54.9	82.3
	4	99.3	149	89.2	134	78.1	117	65.8	98.7	52.6	78.8
	5	93.6	140	84.2	126	73.9	111	62.3	93.4	49.7	74.6
	6	87.0	131	78.5	118	69.0	103	58.2	87.3	46.5	69.7
	7	79.8	120	72.2	108	63.6	95.4	53.8	80.7	42.9	64.4
	8	72.3	108	65.6	98.4	58.0	86.9	49.1	73.7	39.1	58.7
	9	64.6	96.9	58.9	88.3	52.1	78.2	44.3	66.4	35.2	52.9
	10	57.0	85.4	52.1	78.2	46.3	69.5	39.4	59.2	31.4	47.0
	11	49.6	74.4	45.6	68.4	40.7	61.0	34.7	52.1	27.6	41.3
	12	42.5	63.8	39.3	59.0	35.2	52.9	30.2	45.3	23.9	35.9
	13	36.2	54.4	33.5	50.3	30.1	45.2	25.9	38.8	20.5	30.7
	14	31.2	46.9	28.9	43.4	26.0	39.0	22.3	33.4	17.7	26.5
	15	27.2	40.8	25.2	37.8	22.6	34.0	19.4	29.1	15.4	23.1
	16	23.9	35.9	22.1	33.2	19.9	29.8	17.1	25.6	13.5	20.3
	17	21.2	31.8	19.6	29.4	17.6	26.4	15.1	22.7	12.0	18.0
	18	18.9	28.4	17.5	26.2	15.7	23.6	13.5	20.2	10.7	16.0
	19	17.0	25.4	15.7	23.6	14.1	21.2	12.1	18.2	9.59	14.4
	20	15.3	23.0	14.2	21.3	12.7	19.1	10.9	16.4	8.65	13.0
	21	13.9	20.8	12.9	19.3	11.5	17.3	9.91	14.9	7.85	11.8
	22					10.5	15.8	9.03	13.5	7.15	10.7
Properties											
M_n/Ω_b / ϕ_bM_n	kip-ft	10.8	16.2	9.50	14.3	8.03	12.1	6.33	9.51	4.44	6.67
$P_e(KL)^2/10^4$	kip-in.^{2}	202		186		167		144		114	
ASD	LRFD	Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.									
Ω_c = 2.00	ϕ_c = 0.75										

5

COMPOSITE
HSS3

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS3×3×									
		3/8		5/16		1/4		3/16		1/8	
t_{design}, in.		0.349		0.291		0.233		0.174		0.116	
Steel Wt/ft		12.1		10.5		8.78		6.85		4.75	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	88.9	133	79.8	120	69.6	104	58.3	87.5	46.2	69.4
	1	88.1	132	79.1	119	69.0	104	57.8	86.8	45.9	68.8
	2	85.7	129	77.0	115	67.3	101	56.4	84.6	44.7	67.1
	3	81.8	123	73.6	110	64.4	96.7	54.1	81.2	42.9	64.4
	4	76.6	115	69.1	104	60.7	91.0	51.0	76.5	40.5	60.8
	5	70.4	106	63.8	95.7	56.1	84.2	47.3	71.0	37.6	56.4
	6	63.5	95.3	57.8	86.7	51.1	76.6	43.2	64.8	34.3	51.5
	7	56.3	84.4	51.4	77.2	45.6	68.5	38.7	58.1	30.9	46.3
	8	48.9	73.3	45.0	67.5	40.1	60.2	34.2	51.2	27.3	40.9
	9	41.7	62.6	38.6	57.9	34.6	52.0	29.6	44.4	23.7	35.5
	10	34.9	52.4	32.6	48.9	29.4	44.1	25.3	37.9	20.2	30.4
	11	28.9	43.3	27.0	40.5	24.5	36.8	21.2	31.7	17.0	25.5
	12	24.2	36.4	22.7	34.1	20.6	30.9	17.8	26.7	14.3	21.4
	13	20.7	31.0	19.3	29.0	17.5	26.3	15.2	22.7	12.2	18.2
	14	17.8	26.7	16.7	25.0	15.1	22.7	13.1	19.6	10.5	15.7
	15	15.5	23.3	14.5	21.8	13.2	19.8	11.4	17.1	9.13	13.7
	16	13.6	20.5	12.8	19.2	11.6	17.4	10.0	15.0	8.03	12.0
	17	12.1	18.1	11.3	17.0	10.3	15.4	8.86	13.3	7.11	10.7
	18			10.1	15.1	9.15	13.7	7.90	11.9	6.34	9.51
	19							7.09	10.6	5.69	8.54

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	7.46	11.2	6.66	10.0	5.69	8.55	4.53	6.81	3.21	4.82
$P_e(KL)^2/10^4$		kip-in.^{2}	115		107		97.3		84.1		67.5	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 46 ksi
f'_c = 5 ksi

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

5

COMPOSITE
HSS2$^1/_2$–HSS2$^1/_4$

Shape		HSS2$^1/_2$×2$^1/_2$×								HSS2$^1/_4$×2$^1/_4$×	
		$^5/_{16}$		$^1/_4$		$^3/_{16}$		$^1/_8$		$^1/_4$	
t_{design}, in.		0.291		0.233		0.174		0.116		0.233	
Steel Wt/ft		8.40		7.08		5.57		3.90		6.23	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	61.8	92.7	54.1	81.1	45.2	67.8	35.6	53.3	46.7	70.0
	1	61.0	91.4	53.4	80.1	44.7	67.0	35.1	52.7	45.9	68.9
	2	58.5	87.8	51.4	77.1	43.1	64.6	33.9	50.8	43.8	65.6
	3	54.7	82.1	48.2	72.3	40.5	60.8	31.9	47.9	40.4	60.5
	4	49.8	74.7	44.1	66.1	37.2	55.8	29.4	44.1	36.1	54.1
	5	44.1	66.2	39.3	58.9	33.3	49.9	26.4	39.6	31.2	46.8
	6	38.1	57.1	34.1	51.2	29.1	43.7	23.2	34.7	26.1	39.2
	7	31.9	47.9	28.9	43.3	24.8	37.2	19.8	29.8	21.2	31.7
	8	26.1	39.1	23.8	35.8	20.7	31.0	16.6	24.9	16.6	24.9
	9	20.8	31.2	19.2	28.7	16.7	25.1	13.5	20.3	13.1	19.7
	10	16.8	25.3	15.5	23.3	13.6	20.3	11.0	16.4	10.6	15.9
	11	13.9	20.9	12.8	19.2	11.2	16.8	9.06	13.6	8.77	13.2
	12	11.7	17.5	10.8	16.2	9.41	14.1	7.61	11.4	7.37	11.1
	13	10.0	14.9	9.18	13.8	8.02	12.0	6.49	9.73	6.28	9.42
	14	8.59	12.9	7.91	11.9	6.92	10.4	5.59	8.39		
	15			6.89	10.3	6.03	9.04	4.87	7.31		
	16							4.28	6.42		
Properties											
M_n/Ω_b / ϕ_bM_n	kip-ft	4.32	6.49	3.75	5.64	3.03	4.55	2.17	3.27	2.93	4.41
$P_e(KL)^2/10^4$	kip-in.^{2}	55.3		51.0		44.5		36.0		34.8	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

5

COMPOSITE
HSS2$^1/_4$–HSS2

Table 4–16 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Square HSS

F_y = 46 ksi
f'_c = 5 ksi

Shape		HSS2$^1/_4$×2$^1/_4$×				HSS2×2×					
		$^3/_{16}$		$^1/_8$		$^1/_4$		$^3/_{16}$		$^1/_8$	
t_{design}, in.		0.174		0.116		0.233		0.174		0.116	
Steel Wt/ft		4.94		3.47		5.38		4.30		3.04	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	39.1	58.6	30.6	45.9	39.6	59.3	33.2	49.8	25.9	38.9
	1	38.5	57.7	30.2	45.2	38.7	58.1	32.5	48.8	25.4	38.2
	2	36.7	55.1	28.8	43.3	36.4	54.5	30.7	46.0	24.0	36.1
	3	34.0	51.1	26.8	40.2	32.7	49.1	27.8	41.7	21.9	32.8
	4	30.6	45.9	24.2	36.2	28.2	42.4	24.2	36.3	19.2	28.8
	5	26.6	40.0	21.2	31.7	23.4	35.1	20.3	30.4	16.2	24.3
	6	22.5	33.8	18.0	27.0	18.5	27.8	16.3	24.5	13.2	19.8
	7	18.4	27.7	14.8	22.3	14.1	21.1	12.6	18.9	10.3	15.5
	8	14.6	21.9	11.9	17.8	10.8	16.2	9.64	14.5	7.90	11.9
	9	11.6	17.3	9.40	14.1	8.52	12.8	7.62	11.4	6.24	9.37
	10	9.36	14.0	7.61	11.4	6.90	10.4	6.17	9.25	5.06	7.59
	11	7.74	11.6	6.29	9.44	5.70	8.53	5.10	7.65	4.18	6.27
	12	6.50	9.75	5.29	7.93			4.28	6.43	3.51	5.27
	13	5.54	8.31	4.50	6.76						
	14			3.88	5.83						

Properties											
M_n/Ω_b	ϕ_bM_n	kip-ft	2.39	3.60	1.73	2.60	2.21	3.33	1.83	2.75	1.34	2.02
$P_e(KL)^2/10^4$		kip-in.^{2}	30.8		25.1		22.6		20.3		16.6	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 42 ksi
f'_c = 4 ksi

Table 4-17
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

④
COMPOSITE
HSS18.000–
HSS16.000

Shape		HSS18.000×				HSS16.000×							
		0.500		0.375		0.625		0.500		0.438		0.375	
t_{design}, in.		0.465		0.349		0.581		0.465		0.407		0.349	
Steel Wt/ft		93.5		70.7		103		82.8		72.9		62.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	973	1460	853	1280	920	1380	815	1220	763	1140	710	1060
	6	962	1440	844	1270	908	1360	805	1210	753	1130	700	1050
	7	959	1440	840	1260	904	1360	801	1200	749	1120	696	1040
	8	955	1430	836	1250	899	1350	797	1190	745	1120	692	1040
	9	950	1420	832	1250	893	1340	792	1190	740	1110	688	1030
	10	944	1420	827	1240	887	1330	786	1180	735	1100	683	1020
	11	939	1410	822	1230	881	1320	780	1170	729	1090	677	1020
	12	932	1400	816	1220	874	1310	774	1160	723	1080	671	1010
	13	925	1390	809	1210	866	1300	766	1150	716	1070	665	998
	14	918	1380	803	1200	858	1290	759	1140	709	1060	658	987
	15	910	1370	795	1190	849	1270	751	1130	701	1050	651	976
	16	902	1350	788	1180	840	1260	742	1110	693	1040	643	965
	17	893	1340	780	1170	830	1240	733	1100	684	1030	635	952
	18	884	1330	771	1160	820	1230	724	1090	676	1010	626	940
	19	874	1310	762	1140	809	1210	714	1070	666	999	618	926
	20	864	1300	753	1130	798	1200	704	1060	657	985	608	913
	21	854	1280	744	1120	786	1180	694	1040	647	970	599	898
	22	843	1260	734	1100	774	1160	683	1020	636	954	589	884
	23	832	1250	723	1090	762	1140	672	1010	626	938	579	868
	24	821	1230	713	1070	749	1120	660	990	615	922	568	853
	25	809	1210	702	1050	736	1100	648	973	603	905	558	837
	26	797	1200	691	1040	723	1080	636	955	592	888	547	821
	27	784	1180	680	1020	710	1060	624	936	580	871	536	804
	28	772	1160	668	1000	696	1040	612	918	569	853	525	787
	29	759	1140	656	985	682	1020	599	899	557	835	513	770
	30	746	1120	644	967	668	1000	586	879	544	817	502	753
	32	719	1080	620	930	639	959	560	840	520	779	478	718
	34	691	1040	595	892	610	915	534	801	495	742	455	682
	36	663	995	570	854	580	870	507	761	469	704	431	646
	38	635	953	544	816	551	826	480	720	444	666	407	610
	40	606	910	518	777	521	781	454	680	419	628	383	575
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	300	450	228	343	289	435	235	353	207	312	179	269
$P_e(KL)^2/10^4$	kip-in.^{2}	39700		33000		31200		26800		24500		22200	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

④

COMPOSITE
HSS16.000–
HSS14.000

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 4 ksi

Shape		HSS16.000×				HSS14.000×							
		0.312		0.250		0.625		0.500		0.375		0.312	
t_{design}, in.		0.291		0.233		0.581		0.465		0.349		0.291	
Steel Wt/ft		52.3		42.1		89.4		72.2		54.6		45.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	656	984	602	904	760	1140	670	1010	578	868	532	798
	6	647	970	593	890	748	1120	659	988	568	852	522	783
	7	644	965	590	885	743	1120	655	982	565	847	519	778
	8	640	960	587	880	738	1110	650	975	560	841	515	772
	9	635	953	582	874	733	1100	645	968	556	834	510	766
	10	631	946	578	867	726	1090	639	959	551	826	506	758
	11	625	938	573	859	719	1080	633	950	545	817	500	750
	12	620	929	567	851	712	1070	626	939	539	808	494	742
	13	613	920	561	842	703	1060	619	928	532	798	488	732
	14	607	910	555	832	695	1040	611	917	525	788	481	722
	15	600	900	548	822	686	1030	603	904	518	777	474	712
	16	592	889	541	812	676	1010	594	891	510	765	467	701
	17	585	877	534	801	666	999	585	878	502	753	459	689
	18	577	865	526	789	655	983	576	863	493	740	451	677
	19	568	852	518	777	644	966	566	848	484	727	443	664
	20	559	839	509	764	633	949	555	833	475	713	434	651
	21	550	825	501	751	621	931	545	817	466	699	425	638
	22	541	811	492	738	609	913	534	801	456	684	416	624
	23	531	797	483	724	596	894	523	784	446	669	407	610
	24	521	782	473	710	583	875	511	767	436	654	397	595
	25	511	767	464	695	570	856	500	749	426	638	387	581
	26	501	751	454	681	557	836	488	732	415	622	377	566
	27	490	735	444	666	544	816	476	714	404	606	367	551
	28	480	719	434	650	530	795	464	696	394	590	357	536
	29	469	703	423	635	517	775	451	677	383	574	347	521
	30	458	687	413	620	503	754	439	659	372	558	337	505
	32	436	654	392	588	475	712	414	621	350	525	316	474
	34	413	620	371	557	447	670	389	584	328	492	296	444
	36	391	586	350	525	419	629	365	547	306	459	275	413
	38	368	553	329	493	391	587	340	510	285	427	255	383
	40	346	519	308	462	364	547	316	474	264	395	236	354
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	151	226	121	182	220	331	179	268	136	205	115	172
$P_e(KL)^2/10^4$	kip-in.^{2}	19700		17300		19900		17100		14200		12600	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

F_y = 42 ksi
f'_c = 4 ksi

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

④
COMPOSITE
HSS14.000–
HSS10.750

Shape	HSS14.000×		HSS12.750×						HSS10.750×			
	0.250		0.500		0.375		0.250		0.500		0.375	
t_{design}, in.	0.233		0.465		0.349		0.233		0.465		0.349	
Steel Wt/ft	36.7		65.5		49.6		33.4		54.8		41.6	
Design	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)												
0	485	727	585	878	502	753	418	626	459	689	390	585
6	476	714	574	861	492	738	408	612	447	670	379	569
7	472	709	570	854	488	732	405	607	442	663	375	563
8	469	703	565	847	484	726	401	602	437	656	371	556
9	465	697	559	839	479	719	397	595	431	647	366	549
10	460	690	554	830	474	711	392	588	425	638	360	540
11	455	682	547	821	468	702	387	581	418	627	354	531
12	449	674	540	810	462	693	381	572	411	616	348	522
13	443	665	533	799	455	683	376	563	403	604	341	511
14	437	655	525	787	448	672	369	554	394	592	334	500
15	430	645	516	774	441	661	363	544	386	578	326	489
16	423	635	507	761	433	649	356	533	376	565	318	477
17	416	624	498	747	425	637	348	522	367	550	310	465
18	408	612	488	732	416	624	341	511	357	535	301	452
19	400	600	478	717	407	611	333	499	347	520	292	439
20	392	588	468	702	398	597	325	487	336	505	283	425
21	384	575	457	686	389	583	317	475	326	489	274	411
22	375	562	447	670	379	568	308	462	315	473	265	397
23	366	549	435	653	369	554	300	449	304	456	256	383
24	357	535	424	636	359	539	291	436	293	440	246	369
25	348	522	413	619	349	524	282	423	282	424	237	355
26	338	508	401	602	339	508	273	410	271	407	227	341
27	329	494	389	584	329	493	264	396	260	391	218	327
28	320	479	378	566	318	477	255	383	249	374	208	313
29	310	465	366	549	308	462	246	369	239	358	199	299
30	300	451	354	531	297	446	237	356	228	342	190	285
32	281	422	330	495	277	415	220	329	207	310	172	258
34	262	393	307	460	256	384	202	303	187	280	155	232
36	243	365	283	425	236	354	185	278	167	251	138	207
38	225	337	261	391	217	325	169	253	150	225	124	186
40	207	311	239	359	198	297	153	229	135	203	112	168
Properties												
M_n/Ω_b | $\phi_b M_n$ kip-ft	92.6	139	147	221	113	169	76.5	115	103	155	79.2	119
$P_e(KL)^2/10^4$ kip-in.^{2}	11000		12600		10400		8020		7110		5880	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

4

COMPOSITE
HSS10.750–
HSS10.000

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 4 ksi

Shape		HSS10.750×		HSS10.000×									
		0.250		0.625		0.500		0.375		0.312		0.250	
t_{design}, in.		0.233		0.581		0.465		0.349		0.291		0.233	
Steel Wt/ft		28.1		62.6		50.8		38.6		32.3		26.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	319	479	478	716	415	623	351	527	319	478	286	429
	6	310	465	462	694	402	603	340	510	308	462	276	414
	7	306	460	457	685	397	596	336	504	304	457	272	409
	8	302	454	451	676	392	588	331	497	300	450	268	403
	9	298	447	444	666	386	579	326	489	295	443	264	396
	10	293	440	436	655	380	569	321	481	290	435	259	389
	11	288	432	428	642	373	559	314	472	285	427	254	381
	12	282	424	420	629	365	547	308	462	279	418	248	373
	13	276	415	410	615	357	535	301	451	272	408	242	364
	14	270	405	400	601	348	522	294	440	265	398	236	354
	15	263	395	390	585	339	509	286	429	258	387	230	344
	16	257	385	379	569	330	495	278	417	251	376	223	334
	17	249	374	368	552	320	481	270	404	243	365	216	323
	18	242	363	357	535	311	466	261	392	235	353	208	313
	19	234	352	345	518	300	451	252	379	227	341	201	302
	20	227	340	333	500	290	435	244	365	219	329	194	290
	21	219	328	321	482	280	419	235	352	211	316	186	279
	22	211	316	309	463	269	404	225	338	203	304	178	267
	23	203	304	297	445	258	388	216	325	194	291	171	256
	24	195	292	284	427	248	372	207	311	186	279	163	245
	25	187	280	272	408	237	356	198	297	178	266	155	233
	26	179	268	260	390	226	340	189	284	169	254	148	222
	27	171	256	248	372	216	324	180	270	161	242	140	211
	28	163	245	236	354	206	308	171	257	153	230	133	200
	29	155	233	224	336	195	293	163	244	145	218	126	189
	30	148	221	212	319	185	278	154	231	137	206	119	178
	32	133	199	190	285	166	249	137	206	122	183	105	158
	34	118	177	168	253	147	221	122	183	108	162	93.1	140
	36	106	158	150	225	131	197	109	163	96.4	145	83.1	125
	38	94.7	142	135	202	118	177	97.5	146	86.6	130	74.5	112
	40	85.5	128	122	183	106	159	88.0	132	78.1	117	67.3	101

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	54.0	81.2	108	163	88.7	133	68.2	102	57.5	86.4	46.6	70.0
$P_e(KL)^2/10^4$		kip-in.^{2}	4490		6390		5590		4620		4100		3530	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 42 ksi
f'_c = 4 ksi

Table 4-17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

4

COMPOSITE
HSS10.000–
HSS9.625

Shape		HSS10.000×		HSS9.625×									
		0.188		0.500		0.375		0.312		0.250		0.188	
t_{design}, in.		0.174		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		19.7		48.8		37.1		31.1		25.1		19.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	252	378	394	591	332	499	301	452	270	404	237	355
	6	243	364	380	571	321	481	291	436	260	389	228	342
	7	239	359	376	563	317	475	287	430	256	384	224	337
	8	236	354	370	555	312	468	282	424	252	378	221	331
	9	232	347	364	546	307	461	278	416	248	372	217	325
	10	227	341	358	536	301	452	272	409	243	364	212	318
	11	222	333	350	525	295	443	267	400	238	356	207	311
	12	217	326	343	514	289	433	261	391	232	348	202	303
	13	211	317	334	502	282	423	254	381	226	339	197	295
	14	206	308	326	489	274	412	247	371	220	329	191	286
	15	200	299	317	475	267	400	240	360	213	320	185	277
	16	193	290	307	461	259	388	233	349	206	309	178	268
	17	187	280	298	447	251	376	225	338	199	299	172	258
	18	180	270	288	432	242	363	217	326	192	288	166	248
	19	173	260	278	417	233	350	209	314	185	277	159	238
	20	167	250	267	401	225	337	201	302	177	266	152	228
	21	160	239	257	386	216	324	193	290	170	255	145	218
	22	153	229	247	370	207	311	185	278	163	244	139	208
	23	146	219	236	354	198	297	177	265	155	233	132	198
	24	139	208	226	338	189	284	169	253	148	222	125	188
	25	132	198	215	323	180	270	161	241	140	210	119	178
	26	125	188	205	307	172	257	153	229	133	200	112	168
	27	119	178	195	292	163	244	145	217	126	189	106	159
	28	112	168	184	277	154	231	137	205	119	178	99.5	149
	29	106	158	175	262	146	219	129	194	112	168	93.4	140
	30	99.4	149	165	247	138	207	122	183	105	158	87.3	131
	32	87.4	131	146	219	122	183	107	161	92.5	139	76.8	115
	34	77.4	116	129	194	108	162	95.0	143	81.9	123	68.0	102
	36	69.0	104	115	173	96.2	144	84.7	127	73.1	110	60.7	91.0
	38	62.0	92.9	103	155	86.3	129	76.1	114	65.6	98.4	54.4	81.7
	40	55.9	83.9	93.4	140	77.9	117	68.6	103	59.2	88.8	49.1	73.7

Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	35.2	52.9	81.8	123	63.0	94.6	53.2	79.9	43.1	64.8	32.6	49.0
$P_e(KL)^2/10^4$	kip-in.^{2}	2940		4910		4090		3600		3110		2580	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

4

COMPOSITE
HSS8.625

Table 4–17 (continued)
Available Strength in Axial Compression, kips

F_y = 42 ksi
f'_c = 4 ksi

Concrete Filled Round HSS

Shape		HSS8.625×											
		0.625		0.500		0.375		0.322		0.250		0.188	
t_{design}, in.		0.581		0.465		0.349		0.300		0.233		0.174	
Steel Wt/ft		53.5		43.4		33.1		28.6		22.4		17.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length (KL) (ft)	**0**	**391**	587	**339**	508	**284**	426	**261**	391	**228**	343	**199**	299
	6	**375**	562	**324**	487	**272**	408	**250**	374	**218**	327	**190**	285
	7	**369**	553	**319**	479	**268**	402	**246**	368	**214**	322	**187**	280
	8	**362**	543	**314**	470	**263**	395	**241**	362	**210**	316	**183**	274
	9	**355**	532	**307**	461	**258**	387	**236**	354	**206**	309	**179**	268
	10	**347**	520	**300**	450	**252**	378	**231**	346	**201**	301	**174**	261
	11	**338**	507	**293**	439	**246**	368	**225**	337	**196**	293	**169**	254
	12	**329**	493	**285**	427	**239**	358	**219**	328	**190**	285	**164**	246
	13	**319**	478	**276**	415	**232**	348	**212**	318	**184**	276	**159**	238
	14	**308**	463	**268**	401	**224**	336	**205**	308	**178**	267	**153**	230
	15	**298**	447	**258**	388	**216**	325	**198**	297	**171**	257	**147**	221
	16	**287**	430	**249**	373	**208**	313	**190**	286	**165**	247	**141**	212
	17	**275**	413	**239**	359	**200**	300	**183**	274	**158**	237	**135**	202
	18	**264**	396	**229**	344	**192**	288	**175**	263	**151**	227	**129**	193
	19	**252**	378	**219**	329	**184**	275	**167**	251	**144**	216	**123**	184
	20	**241**	361	**209**	314	**175**	263	**160**	239	**137**	206	**116**	174
	21	**229**	343	**199**	299	**167**	250	**152**	228	**130**	195	**110**	165
	22	**217**	326	**189**	284	**158**	237	**144**	216	**123**	185	**104**	156
	23	**206**	308	**179**	269	**150**	225	**136**	204	**116**	174	**97.7**	147
	24	**194**	291	**169**	254	**141**	212	**128**	193	**109**	164	**91.7**	138
	25	**183**	274	**160**	240	**133**	200	**121**	181	**103**	154	**85.8**	129
	26	**172**	258	**150**	225	**125**	188	**114**	170	**96.4**	145	**80.1**	120
	27	**161**	242	**141**	211	**118**	176	**106**	160	**90.0**	135	**74.5**	112
	28	**151**	226	**132**	198	**110**	165	**99.3**	149	**83.8**	126	**69.2**	104
	29	**140**	211	**123**	184	**102**	154	**92.6**	139	**78.1**	117	**64.6**	96.8
	30	**131**	197	**115**	172	**95.7**	144	**86.5**	130	**73.0**	109	**60.3**	90.5
	32	**115**	173	**101**	151	**84.1**	126	**76.1**	114	**64.1**	96.2	**53.0**	79.5
	34	**102**	153	**89.5**	134	**74.5**	112	**67.4**	101	**56.8**	85.2	**47.0**	70.4
	36	**91.1**	137	**79.8**	120	**66.5**	99.7	**60.1**	90.1	**50.7**	76.0	**41.9**	62.8
	38	**81.8**	123	**71.6**	107	**59.7**	89.5	**53.9**	80.9	**45.5**	68.2	**37.6**	56.4
	40	**73.8**	111	**64.6**	97.0	**53.8**	80.8	**48.7**	73.0	**41.0**	61.6	**33.9**	50.9
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	**78.9**	119	**65.0**	97.6	**50.1**	75.3	**43.6**	65.5	**34.4**	51.7	**26.0**	39.2
$P_e(KL)^2/10^4$	kip-in.^{2}	3870		3400		2820		2560		2160		1790	

ASD	**LRFD**
$\Omega_c = 2.00$	$\phi_c = 0.75$

F_y = 42 ksi
f'_c = 4 ksi

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

④
COMPOSITE
HSS7.625–
HSS7.500

Shape		HSS7.625×				HSS7.500×							
		0.375		0.328		0.500		0.375		0.312		0.250	
t_{design}, in.		0.349		0.305		0.465		0.349		0.291		0.233	
Steel Wt/ft		29.1		25.6		37.4		28.6		24.0		19.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	239	359	221	331	280	420	234	351	210	315	186	278
	6	226	339	209	313	265	397	221	331	198	297	175	262
	7	222	333	205	307	259	389	216	324	194	291	171	257
	8	217	325	200	300	253	379	211	316	189	284	167	250
	9	211	316	195	292	246	369	205	308	184	276	162	243
	10	205	307	189	284	239	358	199	299	179	268	157	236
	11	198	298	183	274	231	346	193	289	173	259	152	228
	12	191	287	177	265	223	334	186	279	167	250	146	219
	13	184	276	170	255	214	321	178	268	160	240	140	211
	14	177	265	163	244	205	307	171	256	153	230	134	201
	15	169	253	156	234	196	293	163	245	146	219	128	192
	16	161	241	148	223	186	279	155	233	139	209	122	182
	17	153	229	141	211	176	265	147	221	132	198	115	173
	18	145	217	134	200	167	250	139	209	125	187	109	163
	19	137	205	126	189	157	236	131	197	118	176	102	153
	20	129	193	119	178	148	222	123	185	110	166	95.8	144
	21	121	181	111	167	138	208	116	173	103	155	89.6	134
	22	113	170	104	156	129	194	108	162	96.5	145	83.4	125
	23	106	158	97.2	146	120	180	100	151	89.7	135	77.5	116
	24	98.2	147	90.3	136	112	167	93.1	140	83.2	125	71.6	107
	25	90.9	136	83.6	125	103	154	85.9	129	76.8	115	66.0	99.0
	26	84.0	126	77.3	116	95.1	143	79.5	119	71.0	106	61.0	91.5
	27	77.9	117	71.7	107	88.2	132	73.7	111	65.8	98.7	56.6	84.9
	28	72.4	109	66.6	100	82.0	123	68.5	103	61.2	91.8	52.6	78.9
	29	67.5	101	62.1	93.2	76.4	115	63.9	95.8	57.0	85.6	49.0	73.6
	30	63.1	94.7	58.0	87.1	71.4	107	59.7	89.5	53.3	80.0	45.8	68.7
	32	55.5	83.2	51.0	76.5	62.8	94.2	52.5	78.7	46.9	70.3	40.3	60.4
	34	49.1	73.7	45.2	67.8	55.6	83.4	46.5	69.7	41.5	62.3	35.7	53.5
	36	43.8	65.7	40.3	60.5	49.6	74.4	41.4	62.2	37.0	55.5	31.8	47.7
	38	39.3	59.0	36.2	54.3	44.5	66.8	37.2	55.8	33.2	49.8	28.6	42.8
	40	35.5	53.2	32.7	49.0	40.2	60.3	33.6	50.4	30.0	45.0	25.8	38.7

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft												
		38.8	58.2	34.3	51.5	48.3	72.6	37.4	56.3	31.7	47.7	25.8	38.8
$P_e(KL)^2/10^4$ kip-in.^{2}		1870		1710		2120		1760		1570		1350	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

4

COMPOSITE
HSS7.500–
HSS7.000

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 4 ksi

Shape		HSS7.500×		HSS7.000×							
		0.188		0.500		0.375		0.312		0.250	
t_{design}, in.		0.174		0.465		0.349		0.291		0.233	
Steel Wt/ft		14.7		34.7		26.6		22.3		18.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	160	241	255	383	212	319	190	285	168	252
	6	151	226	239	359	199	298	178	267	157	235
	7	147	221	233	350	194	291	174	261	153	229
	8	144	215	227	341	189	283	169	254	149	223
	9	139	209	220	330	183	275	164	246	144	216
	10	135	202	213	319	177	265	158	237	139	209
	11	130	195	204	307	170	255	152	228	134	200
	12	125	187	196	294	163	245	146	219	128	192
	13	120	180	187	281	156	234	139	209	122	183
	14	114	171	178	267	148	222	133	199	116	174
	15	109	163	169	253	141	211	126	189	110	165
	16	103	154	159	239	133	199	119	178	104	156
	17	97.2	146	150	225	125	188	112	168	97.5	146
	18	91.5	137	141	211	117	176	105	157	91.3	137
	19	85.8	129	131	197	110	164	98.0	147	85.2	128
	20	80.2	120	122	183	102	153	91.2	137	79.2	119
	21	74.7	112	113	170	94.7	142	84.6	127	73.3	110
	22	69.3	104	105	157	87.5	131	78.2	117	67.6	101
	23	64.1	96.2	96.3	144	80.4	121	71.8	108	62.0	93.0
	24	59.0	88.4	88.4	133	73.9	111	66.0	99.0	56.9	85.4
	25	54.3	81.5	81.5	122	68.1	102	60.8	91.2	52.5	78.7
	26	50.2	75.4	75.4	113	62.9	94.4	56.2	84.3	48.5	72.8
	27	46.6	69.9	69.9	105	58.4	87.6	52.1	78.2	45.0	67.5
	28	43.3	65.0	65.0	97.5	54.3	81.4	48.5	72.7	41.8	62.7
	29	40.4	60.6	60.6	90.9	50.6	75.9	45.2	67.8	39.0	58.5
	30	37.7	56.6	56.6	84.9	47.3	70.9	42.2	63.3	36.4	54.7
	32	33.2	49.7	49.7	74.6	41.6	62.3	37.1	55.7	32.0	48.0
	34	29.4	44.1	44.1	66.1	36.8	55.2	32.9	49.3	28.4	42.6
	36	26.2	39.3	39.3	59.0	32.8	49.3	29.3	44.0	25.3	38.0
	38	23.5	35.3	35.3	52.9	29.5	44.2	26.3	39.5	22.7	34.1
	40	21.2	31.8								

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	19.6	29.4	41.7	62.7	32.4	48.7	27.5	41.3	22.4	33.6
$P_e(KL)^2/10^4$ kip-in.^{2}		1110		1670		1400		1250		1080	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 42 ksi
f'_c = 4 ksi

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

4
COMPOSITE
HSS7.000–
HSS6.875

Shape		HSS7.000×				HSS6.875×					
		0.188		0.125		0.500		0.375		0.312	
t_{design}, in.		0.174		0.116		0.465		0.349		0.291	
Steel Wt/ft		13.7		9.19		34.1		26.1		21.9	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	144	217	121	182	249	374	207	311	185	278
	6	135	202	112	168	233	349	194	290	173	260
	7	131	197	109	164	227	341	189	283	169	253
	8	127	191	106	158	221	331	183	275	164	246
	9	123	185	102	153	214	320	178	266	159	238
	10	119	178	97.8	147	206	309	171	257	153	230
	11	114	171	93.5	140	198	297	165	247	147	221
	12	109	163	89.1	134	189	284	158	236	141	212
	13	104	155	84.4	127	181	271	150	225	134	202
	14	98.2	147	79.7	120	172	257	143	214	128	192
	15	92.8	139	74.9	112	162	243	135	203	121	181
	16	87.3	131	70.1	105	153	229	127	191	114	171
	17	81.8	123	65.4	98.1	144	215	120	180	107	160
	18	76.4	115	60.7	91.0	134	201	112	168	100	150
	19	71.0	107	56.1	84.1	125	188	104	157	93.3	140
	20	65.8	98.7	51.6	77.4	116	174	96.9	145	86.6	130
	21	60.7	91.0	47.3	70.9	107	161	89.7	134	80.1	120
	22	55.7	83.6	43.1	64.6	98.9	148	82.6	124	73.8	111
	23	51.0	76.5	39.4	59.1	90.6	136	75.7	114	67.6	101
	24	46.8	70.2	36.2	54.3	83.2	125	69.5	104	62.1	93.1
	25	43.1	64.7	33.4	50.0	76.7	115	64.1	96.1	57.2	85.8
	26	39.9	59.8	30.8	46.3	70.9	106	59.2	88.9	52.9	79.3
	27	37.0	55.5	28.6	42.9	65.7	98.6	54.9	82.4	49.0	73.6
	28	34.4	51.6	26.6	39.9	61.1	91.7	51.1	76.6	45.6	68.4
	29	32.1	48.1	24.8	37.2	57.0	85.5	47.6	71.4	42.5	63.8
	30	30.0	44.9	23.2	34.8	53.3	79.9	44.5	66.7	39.7	59.6
	32	26.3	39.5	20.4	30.5	46.8	70.2	39.1	58.7	34.9	52.4
	34	23.3	35.0	18.0	27.1	41.5	62.2	34.6	52.0	30.9	46.4
	36	20.8	31.2	16.1	24.1	37.0	55.5	30.9	46.3	27.6	41.4
	38	18.7	28.0	14.4	21.7			27.7	41.6	24.8	37.1
	40	16.9	25.3	13.0	19.5						

Properties

M_n/Ω_b	ϕ_bM_n kip-ft	17.0	25.5	11.5	17.3	40.1	60.3	31.2	46.9	26.5	39.8
$P_e(KL)^2/10^4$	kip-in.^{2}	886		685		1570		1310		1170	
ASD	**LRFD**	Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.									
Ω_c = 2.00	ϕ_c = 0.75										

4

COMPOSITE
HSS6.875–
HSS6.625

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 4 ksi

Shape		HSS6.875×				HSS6.625×					
		0.250		0.188		0.500		0.432		0.375	
t_{design}, in.		0.233		0.174		0.465		0.402		0.349	
Steel Wt/ft		17.7		13.4		32.7		28.6		25.1	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	163	245	140	211	237	356	216	323	197	295
	6	152	229	131	196	220	331	200	300	183	274
	7	149	223	127	191	215	322	195	293	178	267
	8	144	216	123	185	208	312	189	284	173	259
	9	140	209	119	179	201	301	183	274	167	250
	10	135	202	115	172	193	290	176	263	160	241
	11	129	194	110	165	185	277	168	252	154	231
	12	124	185	105	157	176	265	161	241	147	220
	13	118	177	99.7	150	168	251	152	229	139	209
	14	112	168	94.4	142	159	238	144	216	132	198
	15	106	158	89.0	133	149	224	136	204	124	186
	16	99.4	149	83.5	125	140	210	128	191	117	175
	17	93.2	140	78.1	117	131	196	119	179	109	164
	18	87.1	131	72.7	109	122	183	111	166	102	152
	19	81.1	122	67.5	101	113	169	103	154	94.1	141
	20	75.1	113	62.3	93.5	104	156	95	142	86.9	130
	21	69.4	104	57.4	86.0	95.7	144	87.3	131	79.9	120
	22	63.8	95.8	52.5	78.7	87.4	131	79.8	120	73.0	110
	23	58.4	87.6	48.0	72.0	79.9	120	73.0	110	66.8	100
	24	53.6	80.4	44.1	66.1	73.4	110	67.0	101	61.4	92.0
	25	49.4	74.1	40.6	60.9	67.7	101	61.8	92.7	56.5	84.8
	26	45.7	68.5	37.6	56.3	62.6	93.8	57.1	85.7	52.3	78.4
	27	42.4	63.6	34.8	52.2	58.0	87.0	53.0	79.5	48.5	72.7
	28	39.4	59.1	32.4	48.6	53.9	80.9	49.3	73.9	45.1	67.6
	29	36.7	55.1	30.2	45.3	50.3	75.4	45.9	68.9	42.0	63.0
	30	34.3	51.5	28.2	42.3	47.0	70.5	42.9	64.4	39.3	58.9
	32	30.2	45.2	24.8	37.2	41.3	61.9	37.7	56.6	34.5	51.8
	34	26.7	40.1	22.0	32.9	36.6	54.9	33.4	50.1	30.6	45.9
	36	23.8	35.8	19.6	29.4	32.6	48.9	29.8	44.7	27.3	40.9
	38	21.4	32.1	17.6	26.4						

Properties

M_n/Ω_b	ϕ_bM_n kip-ft	21.6	32.4	16.4	24.6	37.1	55.7	32.7	49.1	28.8	43.3
$P_e(KL)^2/10^4$	kip-in.^{2}	1010		834		1390		1270		1160	

ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.
Ω_c = 2.00	ϕ_c = 0.75	

F_y = 42 ksi
f'_c = 4 ksi

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

4
COMPOSITE
HSS6.625

Shape		HSS6.625×									
		0.312		0.280		0.250		0.188		0.125	
t_{design}, in.		0.291		0.260		0.233		0.174		0.116	
Steel Wt/ft		21.1		19.0		17.0		12.9		8.69	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	176	264	165	247	155	232	133	199	111	166
	6	164	245	153	230	144	216	123	184	102	153
	7	159	239	149	224	140	210	119	179	98.7	148
	8	154	232	144	217	136	203	116	173	95.3	143
	9	149	224	140	209	131	196	111	167	91.6	137
	10	143	215	134	201	126	189	107	160	87.6	131
	11	137	206	129	193	120	181	102	153	83.4	125
	12	131	197	123	184	115	172	97.1	146	79.0	118
	13	125	187	116	175	109	163	92.0	138	74.5	112
	14	118	177	110	165	103	154	86.8	130	69.9	105
	15	111	167	104	156	97.0	145	81.5	122	65.3	97.9
	16	104	156	97.4	146	90.9	136	76.2	114	60.7	91.0
	17	97.4	146	91.0	136	84.9	127	70.9	106	56.2	84.2
	18	90.7	136	84.7	127	78.9	118	65.7	98.5	51.7	77.6
	19	84.0	126	78.5	118	73.1	110	60.6	90.9	47.4	71.1
	20	77.6	116	72.4	109	67.4	101	55.7	83.5	43.3	64.9
	21	71.3	107	66.6	99.8	61.9	92.8	50.9	76.3	39.2	58.8
	22	65.2	97.8	60.8	91.2	56.4	84.7	46.4	69.5	35.7	53.6
	23	59.6	89.5	55.6	83.4	51.6	77.5	42.4	63.6	32.7	49.1
	24	54.8	82.2	51.1	76.6	47.4	71.1	38.9	58.4	30.0	45.1
	25	50.5	75.7	47.1	70.6	43.7	65.6	35.9	53.8	27.7	41.5
	26	46.7	70.0	43.5	65.3	40.4	60.6	33.2	49.8	25.6	38.4
	27	43.3	64.9	40.4	60.5	37.5	56.2	30.8	46.2	23.7	35.6
	28	40.2	60.4	37.5	56.3	34.8	52.3	28.6	42.9	22.1	33.1
	29	37.5	56.3	35.0	52.5	32.5	48.7	26.7	40.0	20.6	30.9
	30	35.1	52.6	32.7	49.0	30.4	45.5	24.9	37.4	19.2	28.8
	32	30.8	46.2	28.7	43.1	26.7	40.0	21.9	32.9	16.9	25.3
	34	27.3	40.9	25.5	38.2	23.6	35.4	19.4	29.1	15.0	22.4
	36	24.3	36.5	22.7	34.1	21.1	31.6	17.3	26.0	13.3	20.0
	38							15.5	23.3	12.0	18.0
Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	24.5	36.8	22.1	33.2	20.0	30.0	15.2	22.8	10.3	15.5
$P_e(KL)^2/10^4$	kip-in.^{2}	1040		966		897		737		569	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

4

COMPOSITE
HSS6.000

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 4 ksi

Shape		HSS6.000×											
		0.500		0.375		0.312		0.280		0.250		0.188	
t_{design}, in.		0.465		0.349		0.291		0.260		0.233		0.174	
Steel Wt/ft		29.4		22.5		19.0		17.1		15.4		11.7	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	208	312	172	258	153	230	143	215	134	202	115	172
	1	208	311	172	257	153	230	143	214	134	201	114	171
	2	206	309	170	256	152	228	142	213	133	200	113	170
	3	203	305	168	252	150	225	140	210	131	197	112	168
	4	200	300	165	248	147	221	138	206	129	194	110	165
	5	195	293	162	242	144	216	135	202	126	189	107	161
	6	190	285	157	236	140	210	131	196	123	184	104	156
	7	184	276	152	228	136	204	127	190	119	178	101	151
	8	177	266	147	220	131	196	122	183	114	172	96.9	145
	9	170	254	141	211	125	188	117	176	110	164	92.7	139
	10	162	243	134	201	120	179	112	167	105	157	88.3	132
	11	153	230	127	191	113	170	106	159	99.2	149	83.6	125
	12	145	217	120	180	107	161	100	150	93.6	140	78.7	118
	13	136	204	113	169	101	151	93.9	141	87.9	132	73.7	111
	14	127	190	105	158	94.1	141	87.8	132	82.2	123	68.7	103
	15	118	177	98.1	147	87.5	131	81.7	122	76.4	115	63.7	95.6
	16	109	164	90.8	136	81.0	122	75.6	113	70.7	106	58.8	88.1
	17	100	150	83.6	125	74.6	112	69.6	104	65.1	97.6	53.9	80.9
	18	91.8	138	76.6	115	68.4	103	63.8	95.6	59.6	89.4	49.2	73.8
	19	83.6	125	69.9	105	62.3	93.5	58.1	87.2	54.3	81.5	44.7	67.0
	20	75.6	113	63.3	94.9	56.4	84.7	52.6	78.9	49.2	73.7	40.3	60.5
	21	68.6	103	57.4	86.1	51.2	76.8	47.7	71.6	44.6	66.9	36.6	54.9
	22	62.5	93.7	52.3	78.4	46.6	70.0	43.5	65.2	40.6	60.9	33.3	50.0
	23	57.2	85.8	47.8	71.7	42.7	64.0	39.8	59.7	37.2	55.8	30.5	45.7
	24	52.5	78.8	43.9	65.9	39.2	58.8	36.5	54.8	34.1	51.2	28.0	42.0
	25	48.4	72.6	40.5	60.7	36.1	54.2	33.7	50.5	31.5	47.2	25.8	38.7
	26	44.7	67.1	37.4	56.1	33.4	50.1	31.1	46.7	29.1	43.6	23.9	35.8
	28	38.6	57.9	32.3	48.4	28.8	43.2	26.8	40.3	25.1	37.6	20.6	30.9
	30	33.6	50.4	28.1	42.2	25.1	37.6	23.4	35.1	21.8	32.8	17.9	26.9
	32	29.5	44.3	24.7	37.1	22.0	33.1	20.6	30.8	19.2	28.8	15.7	23.6
	34									17.0	25.5	13.9	20.9

Properties

M_n/Ω_b	ϕ_bM_n kip-ft	29.9	45.0	23.4	35.2	19.9	29.9	18.0	27.0	16.2	24.4	12.4	18.6
$P_e(KL)^2/10^4$	kip-in.^{2}	994		831		742		691		645		530	
ASD	**LRFD**	Note: Heavy line indicates *Kl/r* equal to or greater than 200.											
Ω_c = 2.00	ϕ_c = 0.75												

F_y = 42 ksi
f'_c = 4 ksi

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

4

COMPOSITE
HSS6.000–
HSS5.563

Shape		HSS6.000×		HSS5.563×									
		0.125		0.500		0.375		0.258		0.188		0.134	
t_{design}, in.		0.116		0.465		0.349		0.240		0.174		0.124	
Steel Wt/ft		7.85		27.1		20.8		14.6		10.8		7.78	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	94.7	142	188	283	155	233	123	184	102	154	86.6	130
	1	94.4	142	188	282	155	232	122	184	102	153	86.4	130
	2	93.6	140	186	279	154	230	121	182	101	152	85.6	128
	3	92.3	138	183	275	151	227	120	179	99.7	150	84.2	126
	4	90.5	136	180	270	148	222	117	176	97.6	146	82.4	124
	5	88.3	132	175	262	144	217	114	171	95.0	143	80.0	120
	6	85.6	128	169	254	140	210	111	166	92.0	138	77.3	116
	7	82.5	124	163	245	135	202	107	160	88.4	133	74.2	111
	8	79.1	119	156	234	129	193	102	153	84.6	127	70.7	106
	9	75.4	113	148	222	123	184	97.1	146	80.4	121	67.0	100
	10	71.5	107	140	210	116	174	91.8	138	75.9	114	63.1	94.6
	11	67.4	101	132	198	109	164	86.4	130	71.3	107	59.0	88.5
	12	63.2	94.8	123	185	102	153	80.8	121	66.5	99.8	54.8	82.3
	13	58.9	88.4	114	172	95.0	143	75.2	113	61.7	92.6	50.7	76.0
	14	54.6	81.9	106	158	87.9	132	69.5	104	56.9	85.4	46.5	69.8
	15	50.3	75.5	96.9	145	80.7	121	63.9	95.8	52.2	78.3	42.4	63.6
	16	46.1	69.2	88.4	133	73.8	111	58.4	87.5	47.5	71.3	38.4	57.6
	17	42.1	63.1	80.2	120	67.0	101	53.0	79.5	43.1	64.6	34.6	51.9
	18	38.1	57.2	72.2	108	60.5	90.8	47.9	71.8	38.7	58.1	30.9	46.4
	19	34.3	51.4	64.8	97.2	54.3	81.5	43.0	64.4	34.7	52.1	27.7	41.6
	20	30.9	46.4	58.5	87.7	49.0	73.5	38.8	58.2	31.4	47.0	25.0	37.6
	21	28.1	42.1	53.1	79.6	44.5	66.7	35.2	52.8	28.4	42.7	22.7	34.1
	22	25.6	38.4	48.3	72.5	40.5	60.8	32.0	48.1	25.9	38.9	20.7	31.0
	23	23.4	35.1	44.2	66.3	37.1	55.6	29.3	44.0	23.7	35.6	18.9	28.4
	24	21.5	32.2	40.6	60.9	34.0	51.1	26.9	40.4	21.8	32.7	17.4	26.1
	25	19.8	29.7	37.4	56.2	31.4	47.1	24.8	37.2	20.1	30.1	16.0	24.0
	26	18.3	27.5	34.6	51.9	29.0	43.5	22.9	34.4	18.6	27.8	14.8	22.2
	28	15.8	23.7	29.8	44.8	25.0	37.5	19.8	29.7	16.0	24.0	12.8	19.2
	30	13.8	20.6	26.0	39.0	21.8	32.7	17.2	25.8	13.9	20.9	11.1	16.7
	32	12.1	18.1									9.78	14.7
	34	10.7	16.1										
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	8.42	12.7	25.4	38.2	19.9	29.9	14.3	21.4	10.6	15.9	7.69	11.6
$P_e(KL)^2/10^4$	kip-in.^{2}	406		769		644		510		412		328	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

COMPOSITE
HSS5.500–
HSS5.000

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 4 ksi

Shape		HSS5.500×						HSS5.000×					
		0.500		0.375		0.258		0.500		0.375		0.312	
t_{design}, in.		0.465		0.349		0.240		0.465		0.349		0.291	
Steel Wt/ft		26.7		20.5		14.5		24.1		18.5		15.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	186	278	153	230	121	181	164	246	135	202	120	179
	1	185	278	153	229	121	181	163	245	134	201	119	179
	2	183	275	151	227	119	179	161	242	133	199	118	177
	3	181	271	149	223	118	177	158	238	130	196	116	174
	4	177	265	146	219	115	173	154	232	127	191	113	169
	5	172	258	142	213	112	168	149	224	123	185	109	164
	6	166	250	137	206	109	163	143	215	118	177	105	158
	7	160	240	132	198	104	157	137	205	113	169	100	150
	8	153	229	126	190	99.9	150	129	194	107	160	95.0	142
	9	145	218	120	180	95.0	143	121	182	101	151	89.3	134
	10	137	206	114	170	89.8	135	113	170	93.8	141	83.4	125
	11	129	193	107	160	84.4	127	105	157	87.0	130	77.4	116
	12	120	180	99.7	149	78.8	118	96.2	144	80.0	120	71.2	107
	13	111	167	92.5	139	73.1	110	87.7	132	73.1	110	65.1	97.7
	14	103	154	85.4	128	67.5	101	79.4	119	66.3	99.5	59.1	88.6
	15	93.9	141	78.3	117	61.9	92.9	71.3	107	59.7	89.6	53.2	79.9
	16	85.5	128	71.4	107	56.5	84.7	63.6	95.3	53.4	80.1	47.6	71.4
	17	77.4	116	64.7	97.1	51.2	76.8	56.3	84.5	47.3	71.0	42.2	63.4
	18	69.5	104	58.2	87.3	46.1	69.1	50.2	75.3	42.2	63.3	37.7	56.5
	19	62.4	93.5	52.3	78.4	41.3	62.0	45.1	67.6	37.9	56.8	33.8	50.7
	20	56.3	84.4	47.2	70.7	37.3	56.0	40.7	61.0	34.2	51.3	30.5	45.8
	21	51.0	76.6	42.8	64.2	33.8	50.8	36.9	55.3	31.0	46.5	27.7	41.5
	22	46.5	69.8	39.0	58.5	30.8	46.3	33.6	50.4	28.3	42.4	25.2	37.8
	23	42.6	63.8	35.7	53.5	28.2	42.3	30.8	46.1	25.8	38.8	23.1	34.6
	24	39.1	58.6	32.8	49.1	25.9	38.9	28.3	42.4	23.7	35.6	21.2	31.8
	25	36.0	54.0	30.2	45.3	23.9	35.8	26.0	39.1	21.9	32.8	19.5	29.3
	26	33.3	49.9	27.9	41.9	22.1	33.1	24.1	36.1	20.2	30.3	18.1	27.1
	28	28.7	43.1	24.1	36.1	19.0	28.6						
	30			21.0	31.4	16.6	24.9						
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	24.8	37.2	19.4	29.2	13.9	20.9	20.1	30.2	15.9	23.8	13.5	20.4
$P_e(KL)^2/10^4$	kip-in.^{2}	739		619		490		534		449		400	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 42 ksi
f'_c = 4 ksi

Table 4-17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

4

COMPOSITE
HSS5.000–
HSS4.500

Shape		HSS5.000×								HSS4.500×			
		0.258		0.250		0.188		0.125		0.375		0.337	
t_{design}, in.		0.240		0.233		0.174		0.116		0.349		0.313	
Steel Wt/ft		13.1		12.7		9.67		6.51		16.5		15.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	106	159	104	156	87.7	132	71.3	107	117	176	109	163
	1	105	158	104	155	87.4	131	71.0	107	117	175	108	163
	2	104	157	102	154	86.4	130	70.2	105	115	173	107	160
	3	102	154	101	151	84.9	127	68.8	103	113	169	105	157
	4	99.9	150	98.1	147	82.7	124	67.0	100	109	164	101	152
	5	96.8	145	95.0	143	80.0	120	64.7	97.0	105	157	97.4	146
	6	93.0	140	91.3	137	76.9	115	62.0	92.9	99.7	149	92.7	139
	7	88.8	133	87.2	131	73.3	110	58.9	88.3	94.0	141	87.5	131
	8	84.1	126	82.6	124	69.4	104	55.5	83.3	87.9	132	81.8	123
	9	79.1	119	77.7	117	65.2	97.8	52.0	78.0	81.4	122	75.8	114
	10	73.9	111	72.6	109	60.8	91.2	48.3	72.4	74.8	112	69.7	104
	11	68.5	103	67.3	101	56.3	84.5	44.5	66.7	68.0	102	63.4	95.1
	12	63.1	94.7	62.0	92.9	51.8	77.7	40.7	61.0	61.3	92.0	57.2	85.9
	13	57.7	86.5	56.6	85.0	47.3	70.9	36.9	55.3	54.8	82.2	51.2	76.8
	14	52.4	78.5	51.4	77.1	42.8	64.2	33.2	49.8	48.6	72.9	45.4	68.1
	15	47.2	70.8	46.3	69.5	38.5	57.8	29.6	44.5	42.5	63.8	39.8	59.7
	16	42.2	63.3	41.4	62.1	34.4	51.6	26.2	39.3	37.4	56.1	35.0	52.5
	17	37.4	56.1	36.7	55.1	30.4	45.7	23.2	34.8	33.1	49.7	31.0	46.5
	18	33.4	50.1	32.8	49.2	27.2	40.7	20.7	31.1	29.5	44.3	27.6	41.4
	19	30.0	44.9	29.4	44.1	24.4	36.6	18.6	27.9	26.5	39.8	24.8	37.2
	20	27.0	40.6	26.5	39.8	22.0	33.0	16.8	25.2	23.9	35.9	22.4	33.6
	21	24.5	36.8	24.1	36.1	20.0	29.9	15.2	22.8	21.7	32.6	20.3	30.4
	22	22.3	33.5	21.9	32.9	18.2	27.3	13.9	20.8	19.8	29.7	18.5	27.7
	23	20.4	30.7	20.1	30.1	16.6	24.9	12.7	19.0	18.1	27.1	16.9	25.4
	24	18.8	28.2	18.4	27.7	15.3	22.9	11.6	17.5	16.6	24.9	15.5	23.3
	25	17.3	26.0	17.0	25.5	14.1	21.1	10.7	16.1				
	26	16.0	24.0	15.7	23.6	13.0	19.5	9.92	14.9				
	28	13.8	20.7	13.5	20.3	11.2	16.8	8.56	12.8				

Properties												
M_n/Ω_b | $\phi_b M_n$	kip-ft	11.4	17.1	11.1	16.7	8.50	12.8	5.80	8.72	12.6	19.0	11.5	17.3
$P_e(KL)^2/10^4$	kip-in.^{2}	356		348		289		220		314		294	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

4

COMPOSITE
HSS4.500–
HSS4.000

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 4 ksi

Shape		HSS4.500×						HSS4.000×			
		0.237		0.188		0.125		0.313		0.250	
t_{design}, in.		0.220		0.174		0.116		0.291		0.233	
Steel Wt/ft		10.8		8.67		5.85		12.3		10.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	86.7	130	75.4	113	60.7	91.1	88.6	133	76.5	115
	1	86.3	130	75.0	113	60.4	90.7	88.1	132	76.1	114
	2	85.2	128	74.1	111	59.6	89.4	86.7	130	74.8	112
	3	83.3	125	72.4	109	58.2	87.3	84.2	126	72.8	109
	4	80.8	121	70.2	105	56.3	84.5	81.0	121	70.0	105
	5	77.6	116	67.4	101	54.0	80.9	76.9	115	66.5	99.8
	6	73.9	111	64.2	96.3	51.2	76.8	72.3	108	62.5	93.8
	7	69.8	105	60.6	90.9	48.2	72.3	67.2	101	58.1	87.2
	8	65.3	97.9	56.7	85.0	44.9	67.3	61.7	92.5	53.4	80.1
	9	60.6	90.8	52.5	78.8	41.4	62.1	56.0	84.0	48.6	72.8
	10	55.7	83.5	48.3	72.4	37.9	56.8	50.3	75.5	43.6	65.5
	11	50.7	76.1	43.9	65.9	34.3	51.4	44.7	67.0	38.8	58.2
	12	45.8	68.7	39.7	59.5	30.7	46.1	39.2	58.8	34.1	51.1
	13	41.0	61.5	35.5	53.2	27.3	41.0	34.0	51.0	29.6	44.4
	14	36.4	54.5	31.4	47.2	24.0	36.1	29.3	44.0	25.5	38.3
	15	31.9	47.9	27.6	41.4	21.0	31.4	25.5	38.3	22.2	33.3
	16	28.0	42.1	24.2	36.3	18.4	27.6	22.4	33.7	19.5	29.3
	17	24.8	37.3	21.5	32.2	16.3	24.5	19.9	29.8	17.3	26.0
	18	22.2	33.2	19.1	28.7	14.6	21.8	17.7	26.6	15.4	23.2
	19	19.9	29.8	17.2	25.8	13.1	19.6	15.9	23.9	13.9	20.8
	20	17.9	26.9	15.5	23.3	11.8	17.7	14.4	21.5	12.5	18.8
	21	16.3	24.4	14.1	21.1	10.7	16.0	13.0	19.5	11.3	17.0
	22	14.8	22.2	12.8	19.2	9.75	14.6	11.9	17.8	10.3	15.5
	23	13.6	20.4	11.7	17.6	8.92	13.4				
	24	12.5	18.7	10.8	16.2	8.19	12.3				
	25	11.5	17.2	9.92	14.9	7.55	11.3				

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	8.45	12.7	6.83	10.3	4.67	7.02	8.41	12.6	6.94	10.4
$P_e(KL)^2/10^4$		kip-in.^{2}	236		203		156		189		164	
ASD	LRFD		Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$											

F_y = 42 ksi
f'_c = 4 ksi

Table 4–17 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

COMPOSITE
HSS4.000

Shape		HSS4.000×									
		0.237		0.226		0.220		0.188		0.125	
t_{design}, in.		0.220		0.210		0.205		0.174		0.116	
Steel Wt/ft		9.53		9.12		8.89		7.66		5.18	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	73.8	111	71.6	107	70.6	106	63.8	95.7	50.9	76.4
	1	73.4	110	71.2	107	70.2	105	63.5	95.2	50.6	75.9
	2	72.1	108	70.0	105	69.0	103	62.4	93.6	49.7	74.6
	3	70.1	105	68.1	102	67.1	101	60.7	91.0	48.3	72.4
	4	67.4	101	65.5	98.2	64.5	96.7	58.3	87.5	46.3	69.5
	5	64.1	96.2	62.3	93.4	61.3	92.0	55.5	83.2	44.0	65.9
	6	60.3	90.4	58.5	87.8	57.7	86.5	52.1	78.2	41.2	61.8
	7	56.0	84.1	54.4	81.6	53.6	80.4	48.5	72.7	38.2	57.3
	8	51.5	77.3	50.0	75.0	49.3	73.9	44.5	66.8	35.0	52.5
	9	46.8	70.2	45.5	68.2	44.8	67.2	40.5	60.7	31.7	47.5
	10	42.1	63.1	40.9	61.3	40.3	60.4	36.4	54.6	28.3	42.5
	11	37.4	56.1	36.3	54.5	35.8	53.7	32.3	48.5	25.0	37.5
	12	32.9	49.3	31.9	47.9	31.5	47.2	28.4	42.6	21.9	32.8
	13	28.6	42.8	27.7	41.6	27.3	41.0	24.7	37.0	18.8	28.3
	14	24.6	36.9	23.9	35.9	23.6	35.3	21.3	31.9	16.3	24.4
	15	21.4	32.2	20.8	31.2	20.5	30.8	18.5	27.8	14.2	21.2
	16	18.8	28.3	18.3	27.5	18.0	27.0	16.3	24.4	12.4	18.7
	17	16.7	25.0	16.2	24.3	16.0	24.0	14.4	21.6	11.0	16.5
	18	14.9	22.3	14.5	21.7	14.2	21.4	12.9	19.3	9.83	14.7
	19	13.4	20.1	13.0	19.5	12.8	19.2	11.5	17.3	8.82	13.2
	20	12.1	18.1	11.7	17.6	11.5	17.3	10.4	15.6	7.96	11.9
	21	10.9	16.4	10.6	15.9	10.5	15.7	9.45	14.2	7.22	10.8
	22	10.0	15.0	9.68	14.5	9.54	14.3	8.61	12.9	6.58	9.87

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	6.60	9.91	6.33	9.51	6.19	9.31	5.34	8.03	3.67	5.51
$P_e(KL)^2/10^4$		kip-in.^{2}	158		154		151		137		105	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

COMPOSITE
HSS18.000–
HSS16.000

Table 4–18
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS18.000×				HSS16.000×							
		0.500		0.375		0.625		0.500		0.438		0.375	
t_{design}, in.		0.465		0.349		0.581		0.465		0.407		0.349	
Steel Wt/ft		93.5		70.7		103		82.8		72.9		62.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	1080	1620	965	1450	1000	1500	900	1350	849	1270	797	1200
	6	1070	1600	953	1430	988	1480	888	1330	837	1250	785	1180
	7	1060	1600	949	1420	983	1480	883	1320	832	1250	781	1170
	8	1060	1590	944	1420	978	1470	878	1320	827	1240	776	1160
	9	1050	1580	939	1410	972	1460	872	1310	822	1230	771	1160
	10	1050	1570	933	1400	965	1450	866	1300	815	1220	765	1150
	11	1040	1560	926	1390	957	1440	858	1290	808	1210	758	1140
	12	1030	1550	919	1380	949	1420	851	1280	801	1200	751	1130
	13	1020	1540	911	1370	940	1410	842	1260	793	1190	743	1110
	14	1020	1520	903	1350	930	1400	834	1250	784	1180	735	1100
	15	1010	1510	894	1340	920	1380	824	1240	775	1160	726	1090
	16	997	1500	885	1330	910	1360	814	1220	766	1150	716	1070
	17	987	1480	875	1310	898	1350	804	1210	755	1130	707	1060
	18	976	1460	865	1300	887	1330	793	1190	745	1120	696	1040
	19	964	1450	854	1280	874	1310	781	1170	734	1100	686	1030
	20	952	1430	843	1260	862	1290	769	1150	722	1080	675	1010
	21	940	1410	831	1250	848	1270	757	1140	711	1070	663	995
	22	927	1390	819	1230	835	1250	745	1120	698	1050	651	977
	23	914	1370	807	1210	821	1230	732	1100	686	1030	639	959
	24	901	1350	794	1190	806	1210	718	1080	673	1010	627	940
	25	887	1330	781	1170	792	1190	704	1060	660	990	614	921
	26	873	1310	767	1150	777	1160	691	1040	646	969	601	902
	27	858	1290	754	1130	761	1140	676	1010	633	949	588	882
	28	843	1260	740	1110	746	1120	662	993	619	928	575	862
	29	828	1240	726	1090	730	1090	647	971	605	907	561	842
	30	813	1220	711	1070	714	1070	632	949	591	886	548	822
	32	781	1170	682	1020	681	1020	602	904	562	843	520	780
	34	749	1120	652	978	648	972	572	858	533	799	492	738
	36	717	1080	622	933	615	922	541	812	503	755	464	697
	38	684	1030	592	887	581	872	511	766	474	711	437	655
	40	651	976	561	842	548	822	481	721	445	668	409	614

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	300	450	228	343	289	435	235	353	207	312	179	269
$P_e(KL)^2/10^4$	kip-in.^{2}	41000		34300		32100		27600		25300		23000	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

F_y = 42 ksi
f'_c = 5 ksi

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

⑤
COMPOSITE
HSS16.000–
HSS14.000

Shape		HSS16.000×				HSS14.000×							
		0.312		0.250		0.625		0.500		0.375		0.312	
t_{design}, in.		0.291		0.233		0.581		0.465		0.349		0.291	
Steel Wt/ft		52.3		42.1		89.4		72.2		54.6		45.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	745	1120	692	1040	822	1230	734	1100	644	967	599	899
	6	733	1100	681	1020	808	1210	721	1080	632	948	587	881
	7	729	1090	677	1020	802	1200	716	1070	628	942	583	875
	8	725	1090	672	1010	797	1200	711	1070	623	934	578	867
	9	719	1080	667	1000	790	1190	705	1060	617	926	573	859
	10	713	1070	661	992	783	1170	698	1050	611	917	567	850
	11	707	1060	655	982	775	1160	691	1040	604	907	560	841
	12	700	1050	648	972	766	1150	683	1020	597	896	553	830
	13	692	1040	641	961	757	1140	674	1010	589	884	546	819
	14	684	1030	633	949	747	1120	665	998	581	871	538	807
	15	675	1010	624	937	737	1110	656	984	572	858	529	794
	16	666	1000	616	923	726	1090	646	969	563	844	520	780
	17	657	985	606	909	714	1070	635	953	553	830	511	766
	18	647	970	597	895	702	1050	624	936	543	814	501	752
	19	637	955	587	880	690	1030	613	919	532	799	491	737
	20	626	939	576	864	677	1020	601	901	522	782	481	721
	21	615	922	565	848	664	995	589	883	510	766	470	705
	22	603	905	554	831	650	975	576	864	499	748	459	688
	23	592	888	543	814	636	954	563	845	487	731	448	672
	24	580	870	531	797	621	932	550	825	475	713	436	654
	25	567	851	519	779	607	910	537	805	463	695	425	637
	26	555	832	507	761	592	888	523	785	451	676	413	619
	27	542	814	495	743	577	866	510	765	438	657	401	602
	28	529	794	483	724	562	843	496	744	426	639	389	584
	29	517	775	470	706	547	820	482	723	413	620	377	566
	30	503	755	458	687	531	797	468	702	400	601	365	548
	32	477	715	432	649	500	750	440	660	375	563	341	511
	34	450	675	407	610	469	704	412	618	350	525	317	476
	36	424	635	382	572	438	657	384	576	325	487	294	440
	38	397	596	356	535	408	612	357	535	300	451	271	406
	40	371	557	332	498	378	567	330	495	277	415	248	372
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	151	226	121	182	220	331	179	268	136	205	115	172
$P_e(KL)^2/10^4$	kip-in.^{2}	20600		18100		20400		17700		14700		13100	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

5

COMPOSITE
HSS14.000–
HSS10.750

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS14.000×		HSS12.750×						HSS10.750×			
		0.250		0.500		0.375		0.250		0.500		0.375	
t_{design}, in.		0.233		0.465		0.349		0.233		0.465		0.349	
Steel Wt/ft		36.7		65.5		49.6		33.4		54.8		41.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	553	830	637	956	556	835	474	711	495	743	428	642
	6	542	813	624	936	544	816	462	694	481	721	415	623
	7	538	807	619	929	540	810	458	687	476	714	410	616
	8	533	800	614	921	535	802	454	680	470	705	405	608
	9	528	792	608	911	529	794	448	673	464	695	399	599
	10	522	783	601	901	523	784	443	664	456	685	393	589
	11	516	774	593	890	516	774	436	654	449	673	386	579
	12	509	763	585	878	509	763	429	644	440	660	379	568
	13	502	752	577	865	501	751	422	633	431	647	371	556
	14	494	741	568	851	492	739	414	622	422	633	362	543
	15	485	728	558	837	484	725	406	609	412	618	353	530
	16	477	715	548	822	474	711	398	597	402	602	344	516
	17	468	702	537	806	465	697	389	583	391	586	335	502
	18	458	688	526	789	455	682	380	570	380	570	325	487
	19	449	673	515	772	444	666	370	555	368	553	315	472
	20	439	658	503	754	433	650	360	541	357	535	304	456
	21	428	642	491	736	423	634	351	526	345	518	294	441
	22	418	626	478	718	411	617	340	511	333	500	283	425
	23	407	610	466	699	400	600	330	495	321	481	273	409
	24	396	594	453	680	388	583	320	479	309	463	262	393
	25	385	577	440	660	377	565	309	464	297	445	251	377
	26	374	560	427	640	365	547	298	448	285	427	241	361
	27	362	543	414	621	353	529	288	432	272	409	230	345
	28	351	526	400	601	341	512	277	416	260	391	219	329
	29	339	509	387	581	329	494	267	400	248	373	209	313
	30	328	492	374	561	317	476	256	384	237	355	199	298
	32	305	458	347	521	294	440	235	353	214	321	179	268
	34	283	424	321	482	270	406	215	322	192	288	159	239
	36	261	391	296	443	248	372	195	293	171	257	142	213
	38	239	359	271	406	226	339	176	264	153	230	128	191
	40	218	328	247	370	204	307	159	238	139	208	115	173
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	92.6	139	147	221	113	169	76.5	115	103	155	79.2	119
$P_e(KL)^2/10^4$	kip-in.^{2}	11500		13000		10700		8350		7270		6050	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

F_y = 42 ksi
f'_c = 5 ksi

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

5

COMPOSITE
HSS10.750–
HSS10.000

Shape		HSS10.750×		HSS10.000×									
		0.250		0.625		0.500		0.375		0.312		0.250	
t_{design}, in.		0.233		0.581		0.465		0.349		0.291		0.233	
Steel Wt/ft		28.1		62.6		50.8		38.6		32.3		26.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	359	538	507	760	446	669	384	575	352	528	320	480
	6	347	521	490	735	431	647	370	556	339	509	308	462
	7	343	514	484	726	426	639	366	549	335	502	304	455
	8	338	507	477	716	420	630	360	541	330	495	299	448
	9	333	499	470	704	413	620	355	532	324	487	294	440
	10	327	491	461	692	406	609	348	522	318	477	288	432
	11	321	481	452	678	398	597	341	512	312	468	281	422
	12	314	471	443	664	389	584	333	500	305	457	275	412
	13	307	460	432	649	380	570	325	488	297	446	268	401
	14	299	449	422	632	371	556	317	475	289	434	260	390
	15	291	437	410	615	361	541	308	462	281	421	252	378
	16	283	425	398	598	350	526	299	449	272	408	244	366
	17	275	412	386	579	340	509	290	434	263	395	236	354
	18	266	399	374	561	329	493	280	420	254	382	227	341
	19	257	385	361	542	317	476	270	405	245	368	219	328
	20	248	371	348	522	306	459	260	390	236	354	210	315
	21	238	358	335	502	294	441	250	375	226	339	201	302
	22	229	344	322	483	283	424	240	359	217	325	192	288
	23	220	330	308	463	271	406	229	344	207	311	183	275
	24	210	315	295	443	259	389	219	329	198	296	174	262
	25	201	301	282	423	247	371	209	313	188	282	166	248
	26	192	288	269	403	236	354	199	298	179	268	157	236
	27	182	274	256	383	224	336	189	283	170	254	148	223
	28	173	260	243	364	213	319	179	268	160	241	140	210
	29	164	247	230	345	202	303	169	254	152	227	132	198
	30	156	234	218	326	191	286	160	240	143	214	124	186
	32	139	208	193	290	170	254	141	212	126	189	109	163
	34	123	184	171	257	150	225	125	188	112	167	96.5	145
	36	110	164	153	229	134	201	112	167	99.5	149	86.1	129
	38	98.3	147	137	206	120	180	100	150	89.3	134	77.3	116
	40	88.7	133	124	186	109	163	90.4	136	80.6	121	69.7	105

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	54.0	81.2	108	163	88.7	133	68.2	102	57.5	86.4	46.6	70.0
$P_e(KL)^2/10^4$	kip-in.^{2}	4670		6510		5700		4750		4230		3660	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

5

COMPOSITE
HSS10.000–
HSS9.625

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS10.000×		HSS9.625×									
		0.188		0.500		0.375		0.312		0.250		0.188	
t_{design}, in.		0.174		0.465		0.349		0.291		0.233		0.174	
Steel Wt/ft		19.7		48.8		37.1		31.1		25.1		19.0	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	287	430	422	633	362	543	332	498	301	451	269	404
	6	275	413	407	610	349	523	319	479	289	433	258	386
	7	271	407	402	602	344	516	315	472	285	427	254	380
	8	267	400	395	593	339	508	310	465	280	420	249	374
	9	262	392	389	583	333	500	304	456	275	412	244	366
	10	256	384	381	572	327	490	298	447	269	403	238	358
	11	250	375	373	560	319	479	291	437	263	394	233	349
	12	244	365	365	547	312	468	284	426	256	384	226	339
	13	237	355	355	533	304	456	277	415	249	373	219	329
	14	230	345	346	519	296	443	269	403	241	362	212	319
	15	222	334	336	504	287	430	260	391	234	350	205	308
	16	215	322	325	488	278	417	252	378	226	338	198	296
	17	207	310	315	472	268	402	243	365	217	326	190	285
	18	199	298	304	456	259	388	234	351	209	313	182	273
	19	191	286	292	439	249	373	225	337	200	301	174	261
	20	183	274	281	422	239	359	216	324	192	288	166	249
	21	174	262	270	404	229	344	206	310	183	275	158	237
	22	166	249	258	387	219	329	197	296	174	262	150	225
	23	158	237	247	370	209	314	188	282	166	249	142	213
	24	150	225	235	353	199	299	179	268	157	236	134	202
	25	142	213	224	336	189	284	169	254	149	223	127	190
	26	134	201	212	319	180	269	160	241	141	211	119	179
	27	126	189	201	302	170	255	151	227	132	199	112	168
	28	118	178	190	286	160	241	143	214	124	187	105	157
	29	111	166	180	269	151	227	134	201	117	175	97.4	146
	30	104	156	169	254	142	213	126	189	109	163	91.1	137
	32	91.2	137	149	223	125	188	111	166	95.8	144	80.0	120
	34	80.8	121	132	198	111	166	97.9	147	84.9	127	70.9	106
	36	72.0	108	118	177	98.8	148	87.4	131	75.7	114	63.2	94.8
	38	64.6	97.0	106	158	88.7	133	78.4	118	67.9	102	56.8	85.1
	40	58.3	87.5	95.3	143	80.0	120	70.8	106	61.3	92.0	51.2	76.8

Properties

M_n/Ω_b	ϕ_bM_n kip-ft	35.2	52.9	81.8	123	63.0	94.6	53.2	79.9	43.1	64.8	32.6	49.0
$P_e(KL)^2/10^4$	kip-in.^{2}	3070		5010		4200		3720		3230		2690	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

F_y = 42 ksi
f'_c = 5 ksi

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

COMPOSITE
HSS8.625

Shape		HSS8.625×											
		0.625		0.500		0.375		0.322		0.250		0.188	
t_{design}, in.		0.581		0.465		0.349		0.300		0.233		0.174	
Steel Wt/ft		53.5		43.4		33.1		28.6		22.4		17.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	412	618	361	541	308	462	285	427	253	380	225	337
	6	394	591	345	517	294	441	272	408	241	361	213	320
	7	387	581	339	509	289	434	267	401	237	355	209	314
	8	380	570	333	499	284	425	262	393	232	348	205	307
	9	372	558	326	489	277	416	256	384	227	340	200	300
	10	363	545	318	477	271	406	250	375	221	331	194	291
	11	354	531	310	465	264	395	243	365	215	322	188	283
	12	344	516	301	452	256	384	236	354	208	312	182	273
	13	333	499	292	438	248	372	228	343	201	301	176	263
	14	322	483	282	423	240	359	221	331	194	290	169	253
	15	310	465	272	408	231	346	212	319	186	279	162	243
	16	298	448	261	392	222	333	204	306	178	267	155	232
	17	286	429	251	376	213	319	195	293	170	256	147	221
	18	274	411	240	360	203	305	187	280	162	244	140	210
	19	261	392	229	344	194	291	178	267	154	232	133	199
	20	249	373	218	327	184	277	169	253	146	220	125	188
	21	236	354	207	311	175	263	160	240	138	208	118	177
	22	224	336	196	294	166	249	151	227	130	196	111	166
	23	211	317	186	278	156	235	143	214	123	184	104	156
	24	199	299	175	262	147	221	134	201	115	173	96.9	145
	25	187	281	164	247	138	208	126	189	108	161	90.2	135
	26	176	263	154	231	130	194	118	177	100	150	83.5	125
	27	164	246	144	216	121	182	110	165	93.1	140	77.4	116
	28	153	229	134	202	113	169	102	153	86.5	130	72.0	108
	29	142	214	125	188	105	157	95.2	143	80.7	121	67.1	101
	30	133	200	117	176	98.1	147	89.0	133	75.4	113	62.7	94.1
	32	117	175	103	154	86.2	129	78.2	117	66.3	99.4	55.1	82.7
	34	104	155	91.1	137	76.4	115	69.3	104	58.7	88.0	48.8	73.3
	36	92.4	139	81.3	122	68.1	102	61.8	92.7	52.4	78.5	43.6	65.3
	38	83.0	124	72.9	109	61.1	91.7	55.4	83.2	47.0	70.5	39.1	58.6
	40	74.9	112	65.8	98.7	55.2	82.8	50.0	75.1	42.4	63.6	35.3	52.9
Properties													
M_n/Ω_b | $\phi_b M_n$	kip-ft	78.9	119	65.0	97.6	50.1	75.3	43.6	65.5	34.4	51.7	26.0	39.2
$P_e(KL)^2/10^4$	kip-in.^{2}	3930		3460		2890		2640		2230		1860	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

5

COMPOSITE
HSS7.625–
HSS7.500

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS7.625×				HSS7.500×							
		0.375		0.328		0.500		0.375		0.312		0.250	
t_{design}, in.		0.349		0.305		0.465		0.349		0.291		0.233	
Steel Wt/ft		29.1		25.6		37.4		28.6		24.0		19.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	257	386	239	359	296	445	251	376	228	342	204	306
	6	242	364	225	338	279	419	236	354	214	321	191	287
	7	237	356	221	331	273	410	231	347	209	314	187	281
	8	232	347	215	323	266	400	225	338	204	306	182	273
	9	225	338	209	314	259	388	219	329	198	298	177	265
	10	218	328	203	304	251	376	212	318	192	288	171	257
	11	211	317	196	294	242	363	205	307	185	278	165	247
	12	203	305	189	283	233	350	197	296	178	267	158	237
	13	195	293	181	272	224	335	189	283	171	256	151	227
	14	187	280	173	260	214	321	181	271	163	245	144	217
	15	178	267	165	248	204	306	172	258	155	233	137	206
	16	170	254	157	236	193	290	163	245	147	221	130	195
	17	161	241	149	223	183	275	154	232	139	209	123	184
	18	152	228	141	211	173	259	146	219	131	197	115	173
	19	143	214	132	199	162	244	137	205	123	185	108	162
	20	134	201	124	186	152	228	128	192	115	173	101	151
	21	126	188	116	174	142	213	120	180	108	162	93.8	141
	22	117	176	108	162	132	199	111	167	100	150	86.9	130
	23	109	163	101	151	123	184	103	155	92.7	139	80.3	120
	24	101	151	93	140	113	170	95.3	143	85.5	128	73.8	111
	25	92.9	139	85.7	129	105	157	87.8	132	78.8	118	68.0	102
	26	85.9	129	79.2	119	96.6	145	81.2	122	72.8	109	62.9	94.3
	27	79.6	119	73.5	110	89.6	134	75.3	113	67.5	101	58.3	87.4
	28	74.0	111	68.3	102	83.3	125	70.0	105	62.8	94.2	54.2	81.3
	29	69.0	104	63.7	95.5	77.7	116	65.3	97.9	58.5	87.8	50.5	75.8
	30	64.5	96.8	59.5	89.3	72.6	109	61.0	91.5	54.7	82.0	47.2	70.8
	32	56.7	85.0	52.3	78.5	63.8	95.7	53.6	80.4	48.1	72.1	41.5	62.2
	34	50.2	75.3	46.3	69.5	56.5	84.7	47.5	71.2	42.6	63.9	36.8	55.1
	36	44.8	67.2	41.3	62.0	50.4	75.6	42.3	63.5	38.0	57.0	32.8	49.2
	38	40.2	60.3	37.1	55.6	45.2	67.8	38.0	57.0	34.1	51.1	29.4	44.1
	40	36.3	54.4	33.5	50.2	40.8	61.2	34.3	51.5	30.8	46.2	26.6	39.8
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	38.8	58.2	34.3	51.5	48.3	72.6	37.4	56.3	31.7	47.7	25.8	38.8
$P_e(KL)^2/10^4$	kip-in.^{2}	1900		1760		2150		1800		1610		1400	
ASD	LRFD												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

F_y = 42 ksi
f'_c = 5 ksi

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

5

COMPOSITE
HSS7.500–
HSS7.000

Shape		HSS7.500×		HSS7.000×							
		0.188		0.500		0.375		0.312		0.250	
t_{design}, in.		0.174		0.465		0.349		0.291		0.233	
Steel Wt/ft		14.7		34.7		26.6		22.3		18.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	180	269	269	404	227	341	206	308	184	275
	6	168	252	251	377	212	318	192	288	171	256
	7	164	246	245	368	207	310	187	280	167	250
	8	159	239	238	357	201	301	182	272	162	242
	9	154	231	231	346	194	292	176	264	156	234
	10	149	223	222	334	188	281	169	254	150	226
	11	143	215	214	320	180	270	163	244	144	216
	12	137	206	204	307	172	258	156	233	138	207
	13	131	196	195	292	164	246	148	222	131	197
	14	124	187	185	278	156	234	141	211	124	186
	15	118	177	175	263	147	221	133	199	117	176
	16	111	167	165	248	139	208	125	188	110	165
	17	105	157	155	232	130	196	117	176	103	155
	18	97.9	147	145	217	122	183	110	165	96.1	144
	19	91.3	137	135	202	114	170	102	153	89.3	134
	20	84.9	127	125	188	105	158	94.7	142	82.6	124
	21	78.6	118	116	174	97.4	146	87.4	131	76.1	114
	22	72.6	109	107	160	89.7	135	80.4	121	69.7	105
	23	66.5	99.8	97.7	147	82.1	123	73.6	110	63.8	95.7
	24	61.1	91.7	89.7	135	75.4	113	67.6	101	58.6	87.9
	25	56.3	84.5	82.7	124	69.5	104	62.3	93.4	54.0	81.0
	26	52.1	78.1	76.4	115	64.2	96.3	57.6	86.4	49.9	74.9
	27	48.3	72.4	70.9	106	59.6	89.3	53.4	80.1	46.3	69.4
	28	44.9	67.4	65.9	98.9	55.4	83.1	49.7	74.5	43.0	64.6
	29	41.9	62.8	61.5	92.2	51.6	77.4	46.3	69.4	40.1	60.2
	30	39.1	58.7	57.4	86.1	48.2	72.4	43.3	64.9	37.5	56.2
	32	34.4	51.6	50.5	75.7	42.4	63.6	38.0	57.0	33.0	49.4
	34	30.5	45.7	44.7	67.1	37.6	56.3	33.7	50.5	29.2	43.8
	36	27.2	40.7	39.9	59.8	33.5	50.3	30.0	45.1	26.0	39.1
	38	24.4	36.6	35.8	53.7	30.1	45.1	27.0	40.4	23.4	35.1
	40	22.0	33.0								

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	19.6	29.4	41.7	62.7	32.4	48.7	27.5	41.3	22.4	33.6
$P_e(KL)^2/10^4$	kip-in.^{2}	1160		1700		1430		1280		1110	
ASD	**LRFD**	Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

5

COMPOSITE
HSS7.000–
HSS6.875

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS7.000×				HSS6.875×					
		0.188		0.125		0.500		0.375		0.312	
t_{design}, in.		0.174		0.116		0.465		0.349		0.291	
Steel Wt/ft		13.7		9.19		34.1		26.1		21.9	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	161	241	138	207	263	394	221	332	200	300
	6	149	224	127	191	244	367	206	309	186	279
	7	145	218	123	185	238	357	201	301	182	272
	8	141	211	119	179	231	347	195	292	176	264
	9	136	203	114	172	224	335	188	283	170	255
	10	130	195	109	164	215	323	181	272	164	246
	11	125	187	104	156	207	310	174	261	157	236
	12	119	178	98.8	148	197	296	166	249	150	225
	13	113	169	93.2	140	188	282	158	237	143	214
	14	106	159	87.5	131	178	267	150	225	135	203
	15	99.9	150	81.8	123	168	252	142	212	127	191
	16	93.6	140	76.1	114	158	237	133	200	120	180
	17	87.3	131	70.5	106	148	222	125	187	112	168
	18	81.1	122	65.0	97.5	138	207	116	174	104	157
	19	74.9	112	59.6	89.4	128	193	108	162	97.0	146
	20	69.0	104	54.4	81.7	119	178	99.9	150	89.7	135
	21	63.3	94.9	49.4	74.1	110	164	92.1	138	82.6	124
	22	57.7	86.5	45.0	67.5	100	151	84.4	127	75.6	113
	23	52.8	79.1	41.2	61.8	91.9	138	77.2	116	69.2	104
	24	48.5	72.7	37.8	56.7	84.4	127	70.9	106	63.6	95.3
	25	44.7	67.0	34.9	52.3	77.8	117	65.3	98.0	58.6	87.9
	26	41.3	61.9	32.2	48.3	71.9	108	60.4	90.6	54.2	81.2
	27	38.3	57.4	29.9	44.8	66.7	100	56.0	84.0	50.2	75.3
	28	35.6	53.4	27.8	41.7	62.0	93.0	52.1	78.1	46.7	70.0
	29	33.2	49.8	25.9	38.9	57.8	86.7	48.6	72.8	43.5	65.3
	30	31.0	46.5	24.2	36.3	54.0	81.0	45.4	68.1	40.7	61.0
	32	27.3	40.9	21.3	31.9	47.5	71.2	39.9	59.8	35.8	53.6
	34	24.1	36.2	18.8	28.3	42.1	63.1	35.3	53.0	31.7	47.5
	36	21.5	32.3	16.8	25.2	37.5	56.3	31.5	47.3	28.3	42.4
	38	19.3	29.0	15.1	22.6			28.3	42.4	25.4	38.0
	40	17.4	26.2	13.6	20.4						
Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	17.0	25.5	11.5	17.3	40.1	60.3	31.2	46.9	26.5	39.8
$P_e(KL)^2/10^4$	kip-in.^{2}	916		716		1600		1340		1200	
ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 42 ksi
f'_c = 5 ksi

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

5

COMPOSITE
HSS6.875–
HSS6.625

Shape		HSS6.875×				HSS6.625×					
		0.250		0.188		0.500		0.432		0.375	
t_{design}, in.		0.233		0.174		0.465		0.402		0.349	
Steel Wt/ft		17.7		13.4		32.7		28.6		25.1	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	179	268	156	235	249	374	228	342	210	315
	6	166	249	145	217	231	347	211	317	194	292
	7	162	242	141	211	225	337	206	308	189	284
	8	157	235	136	204	218	326	199	299	183	275
	9	151	227	131	197	210	315	192	288	177	265
	10	145	218	126	189	201	302	184	277	170	254
	11	139	209	120	180	193	289	176	264	162	243
	12	133	199	114	171	183	275	168	252	154	231
	13	126	189	108	162	174	261	159	239	146	219
	14	119	179	102	153	164	246	150	225	138	207
	15	112	168	95.6	143	154	231	141	212	130	195
	16	105	158	89.4	134	144	217	132	198	121	182
	17	98.4	148	83.2	125	135	202	123	185	113	170
	18	91.5	137	77.0	116	125	187	114	171	105	157
	19	84.8	127	71.1	107	115	173	106	158	97.0	145
	20	78.3	117	65.2	97.9	106	159	97.2	146	89.2	134
	21	71.9	108	59.6	89.3	97.2	146	89.0	134	81.7	123
	22	65.6	98.5	54.3	81.4	88.6	133	81.1	122	74.4	112
	23	60.1	90.1	49.7	74.5	81.0	122	74.2	111	68.1	102
	24	55.2	82.7	45.6	68.4	74.4	112	68.1	102	62.5	93.8
	25	50.8	76.3	42.0	63.0	68.6	103	62.8	94.2	57.6	86.4
	26	47.0	70.5	38.9	58.3	63.4	95.1	58.1	87.1	53.3	79.9
	27	43.6	65.4	36.0	54.0	58.8	88.2	53.8	80.8	49.4	74.1
	28	40.5	60.8	33.5	50.3	54.7	82.0	50.1	75.1	45.9	68.9
	29	37.8	56.7	31.2	46.8	51.0	76.5	46.7	70.0	42.8	64.2
	30	35.3	53.0	29.2	43.8	47.6	71.4	43.6	65.4	40.0	60.0
	32	31.0	46.5	25.7	38.5	41.9	62.8	38.3	57.5	35.2	52.8
	34	27.5	41.2	22.7	34.1	37.1	55.6	34.0	50.9	31.2	46.7
	36	24.5	36.8	20.3	30.4	33.1	49.6	30.3	45.4	27.8	41.7
	38	22.0	33.0	18.2	27.3						
Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	21.6	32.4	16.4	24.6	37.1	55.7	32.7	49.1	28.8	43.3
$P_e(KL)^2/10^4$	kip-in.^{2}	1040		863		1410		1290		1180	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

5

COMPOSITE
HSS6.625

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS6.625×									
		0.312		0.280		0.250		0.188		0.125	
t_{design}, in.		0.291		0.260		0.233		0.174		0.116	
Steel Wt/ft		21.1		19.0		17.0		12.9		8.69	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	190	285	179	268	169	254	148	221	126	189
	6	176	263	165	248	156	234	136	203	115	172
	7	171	256	161	241	152	228	132	197	111	167
	8	165	248	156	233	147	220	127	191	107	160
	9	159	239	150	225	141	212	122	183	102	154
	10	153	229	144	216	136	203	117	175	97.5	146
	11	146	219	137	206	129	194	111	167	92.4	139
	12	139	209	131	196	123	184	105	158	87.1	131
	13	132	198	124	186	116	175	99.4	149	81.7	123
	14	124	186	117	175	110	164	93.4	140	76.3	114
	15	117	175	110	164	103	154	87.2	131	70.8	106
	16	109	164	102	154	96.0	144	81.2	122	65.4	98.1
	17	102	153	95.4	143	89.2	134	75.1	113	60.1	90.1
	18	94.3	141	88.4	133	82.6	124	69.2	104	54.9	82.4
	19	87.1	131	81.6	122	76.1	114	63.5	95.3	50.0	74.9
	20	80.0	120	74.9	112	69.8	105	58.0	87.0	45.1	67.7
	21	73.2	110	68.5	103	63.7	95.5	52.6	78.9	40.9	61.4
	22	66.7	100	62.4	93.6	58.0	87.0	47.9	71.9	37.3	55.9
	23	61.0	91.5	57.1	85.6	53.1	79.6	43.8	65.8	34.1	51.2
	24	56.0	84.1	52.4	78.6	48.7	73.1	40.3	60.4	31.3	47.0
	25	51.6	77.5	48.3	72.4	44.9	67.4	37.1	55.7	28.9	43.3
	26	47.7	71.6	44.7	67.0	41.5	62.3	34.3	51.5	26.7	40.0
	27	44.3	66.4	41.4	62.1	38.5	57.8	31.8	47.7	24.8	37.1
	28	41.2	61.8	38.5	57.8	35.8	53.7	29.6	44.4	23.0	34.5
	29	38.4	57.6	35.9	53.8	33.4	50.1	27.6	41.4	21.5	32.2
	30	35.9	53.8	33.5	50.3	31.2	46.8	25.8	38.7	20.1	30.1
	32	31.5	47.3	29.5	44.2	27.4	41.1	22.7	34.0	17.6	26.4
	34	27.9	41.9	26.1	39.2	24.3	36.4	20.1	30.1	15.6	23.4
	36	24.9	37.4	23.3	34.9	21.7	32.5	17.9	26.8	13.9	20.9
	38							16.1	24.1	12.5	18.7

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	24.5	36.8	22.1	33.2	20.0	30.0	15.2	22.8	10.3	15.5
$P_e(KL)^2/10^4$	kip-in.^{2}	1060		991		922		762		593	
ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 42 ksi
f'_c = 5 ksi

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

⑤
COMPOSITE
HSS6.000

Shape		HSS6.000×											
		0.500		0.375		0.312		0.280		0.250		0.188	
t_{design}, in.		0.465		0.349		0.291		0.260		0.233		0.174	
Steel Wt/ft		29.4		22.5		19.0		17.1		15.4		11.7	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	218	327	183	274	164	247	154	232	146	219	126	190
	1	217	326	182	273	164	246	154	231	145	218	126	189
	2	215	323	181	271	163	244	153	229	144	216	125	188
	3	213	319	178	267	161	241	151	226	142	214	123	185
	4	209	313	175	263	158	236	148	222	140	210	121	181
	5	204	306	171	257	154	231	145	217	136	205	118	177
	6	198	297	166	249	150	224	141	211	133	199	114	172
	7	192	287	161	241	145	217	136	204	128	192	110	165
	8	184	276	155	232	139	209	131	196	123	185	106	159
	9	176	264	148	222	133	199	125	187	118	176	101	151
	10	168	252	141	211	127	190	119	178	112	168	95.7	144
	11	159	238	133	200	120	180	112	169	106	159	90.2	135
	12	149	224	125	188	113	169	106	159	99.5	149	84.6	127
	13	140	210	118	176	106	158	99.0	149	93.1	140	78.9	118
	14	131	196	110	164	98.4	148	92.2	138	86.7	130	73.2	110
	15	121	181	102	152	91.2	137	85.4	128	80.3	120	67.5	101
	16	112	167	93.7	141	84.1	126	78.7	118	73.9	111	61.9	92.9
	17	102	154	86.0	129	77.1	116	72.2	108	67.7	102	56.5	84.7
	18	93.4	140	78.5	118	70.4	106	65.8	98.7	61.7	92.6	51.2	76.8
	19	84.8	127	71.3	107	63.8	95.8	59.7	89.5	55.9	83.8	46.1	69.1
	20	76.5	115	64.3	96.5	57.6	86.4	53.8	80.8	50.4	75.7	41.6	62.4
	21	69.4	104	58.4	87.5	52.3	78.4	48.8	73.3	45.7	68.6	37.7	56.6
	22	63.2	94.8	53.2	79.8	47.6	71.4	44.5	66.8	41.7	62.5	34.4	51.6
	23	57.9	86.8	48.6	73.0	43.6	65.4	40.7	61.1	38.1	57.2	31.5	47.2
	24	53.1	79.7	44.7	67.0	40.0	60.0	37.4	56.1	35.0	52.5	28.9	43.3
	25	49.0	73.4	41.2	61.8	36.9	55.3	34.5	51.7	32.3	48.4	26.6	39.9
	26	45.3	67.9	38.1	57.1	34.1	51.1	31.9	47.8	29.8	44.8	24.6	36.9
	28	39.0	58.6	32.8	49.2	29.4	44.1	27.5	41.2	25.7	38.6	21.2	31.8
	30	34.0	51.0	28.6	42.9	25.6	38.4	23.9	35.9	22.4	33.6	18.5	27.7
	32	29.9	44.8	25.1	37.7	22.5	33.8	21.0	31.6	19.7	29.6	16.2	24.4
	34									17.5	26.2	14.4	21.6

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	29.9	45.0	23.4	35.2	19.9	29.9	18.0	27.0	16.2	24.4	12.4	18.6
$P_e(KL)^2/10^4$	kip-in.^{2}	1010		845		757		707		662		546	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

COMPOSITE
HSS6.000–
HSS5.563

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS6.000×		HSS5.563×									
		0.125		0.500		0.375		0.258		0.188		0.134	
t_{design}, in.		0.116		0.465		0.349		0.240		0.174		0.124	
Steel Wt/ft		7.85		27.1		20.8		14.6		10.8		7.78	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	107	161	196	295	164	246	132	199	113	169	97.2	146
	1	107	160	196	294	164	246	132	198	112	168	96.9	145
	2	106	159	194	291	162	243	131	196	111	167	95.9	144
	3	104	156	191	287	160	240	129	193	109	164	94.2	141
	4	102	153	187	281	156	235	126	189	107	160	92.0	138
	5	99.2	149	182	273	152	228	123	184	104	156	89.2	134
	6	95.9	144	176	264	147	221	119	178	100	151	85.9	129
	7	92.2	138	169	254	142	212	114	171	96.3	144	82.1	123
	8	88.1	132	162	242	135	203	109	163	91.7	138	78.0	117
	9	83.6	125	153	230	128	193	103	155	86.9	130	73.6	110
	10	78.9	118	145	217	121	182	97.5	146	81.8	123	68.9	103
	11	74.0	111	136	204	114	171	91.4	137	76.5	115	64.2	96.2
	12	69.0	104	127	190	106	159	85.2	128	71.0	107	59.3	88.9
	13	63.9	95.9	117	176	98.4	148	78.9	118	65.6	98.4	54.4	81.6
	14	58.9	88.3	108	162	90.7	136	72.7	109	60.1	90.2	49.6	74.4
	15	53.9	80.9	98.9	148	83.1	125	66.5	99.7	54.8	82.2	44.9	67.3
	16	49.0	73.6	90.0	135	75.7	113	60.5	90.7	49.6	74.5	40.4	60.5
	17	44.3	66.5	81.4	122	68.5	103	54.7	82.0	44.7	67.0	36.0	53.9
	18	39.8	59.6	73.0	109	61.5	92.2	49.0	73.5	39.9	59.8	32.1	48.1
	19	35.7	53.5	65.5	98.3	55.2	82.7	44.0	66.0	35.8	53.7	28.8	43.2
	20	32.2	48.3	59.1	88.7	49.8	74.7	39.7	59.5	32.3	48.4	26.0	39.0
	21	29.2	43.8	53.6	80.4	45.2	67.7	36.0	54.0	29.3	43.9	23.6	35.3
	22	26.6	39.9	48.9	73.3	41.1	61.7	32.8	49.2	26.7	40.0	21.5	32.2
	23	24.4	36.5	44.7	67.1	37.6	56.5	30.0	45.0	24.4	36.6	19.6	29.5
	24	22.4	33.6	41.1	61.6	34.6	51.9	27.6	41.3	22.4	33.6	18.0	27.1
	25	20.6	30.9	37.8	56.8	31.9	47.8	25.4	38.1	20.7	31.0	16.6	24.9
	26	19.1	28.6	35.0	52.5	29.5	44.2	23.5	35.2	19.1	28.7	15.4	23.1
	28	16.4	24.6	30.2	45.3	25.4	38.1	20.2	30.4	16.5	24.7	13.3	19.9
	30	14.3	21.5	26.3	39.4	22.1	33.2	17.6	26.5	14.4	21.5	11.5	17.3
	32	12.6	18.9									10.1	15.2
	34	11.1	16.7										
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	8.42	12.7	25.4	38.2	19.9	29.9	14.3	21.4	10.6	15.9	7.69	11.6
$P_e(KL)^2/10^4$	kip-in.^{2}	423		776		654		521		425		341	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.											
$\Omega_c = 2.00$	$\phi_c = 0.75$												

F_y = 42 ksi
f'_c = 5 ksi

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

COMPOSITE
HSS5.500–
HSS5.000

Shape		HSS5.500×						HSS5.000×					
		0.500		0.375		0.258		0.500		0.375		0.312	
t_{design}, in.		0.465		0.349		0.240		0.465		0.349		0.291	
Steel Wt/ft		26.7		20.5		14.5		24.1		18.5		15.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	193	290	162	242	130	195	170	255	142	212	127	190
	1	193	289	161	242	130	195	169	254	141	212	126	190
	2	191	287	160	239	129	193	167	251	139	209	125	187
	3	188	282	157	236	127	190	164	246	137	205	123	184
	4	184	276	154	231	124	186	160	240	133	200	119	179
	5	179	268	150	224	120	181	155	232	129	193	115	173
	6	173	259	145	217	116	175	148	222	124	186	111	166
	7	166	249	139	208	112	168	141	212	118	177	106	158
	8	158	238	133	199	107	160	133	200	111	167	99.8	150
	9	150	225	126	189	101	152	125	187	105	157	93.6	140
	10	142	212	119	178	95.3	143	116	174	97.3	146	87.2	131
	11	133	199	111	167	89.2	134	107	161	90.0	135	80.6	121
	12	123	185	103	155	83.0	124	98.4	148	82.5	124	73.9	111
	13	114	171	95.8	144	76.7	115	89.5	134	75.1	113	67.3	101
	14	105	157	88.1	132	70.5	106	80.7	121	67.9	102	60.8	91.2
	15	95.8	144	80.5	121	64.4	96.6	72.3	108	60.9	91.4	54.5	81.8
	16	87.0	131	73.1	110	58.4	87.7	64.2	96.2	54.1	81.2	48.5	72.7
	17	78.5	118	66.0	99.1	52.7	79.1	56.8	85.2	48.0	71.9	43.0	64.4
	18	70.2	105	59.1	88.7	47.1	70.7	50.7	76.0	42.8	64.2	38.3	57.5
	19	63.0	94.5	53.1	79.6	42.3	63.5	45.5	68.2	38.4	57.6	34.4	51.6
	20	56.9	85.3	47.9	71.8	38.2	57.3	41.1	61.6	34.7	52.0	31.0	46.6
	21	51.6	77.4	43.4	65.2	34.6	51.9	37.2	55.9	31.4	47.1	28.2	42.2
	22	47.0	70.5	39.6	59.4	31.6	47.3	33.9	50.9	28.6	43.0	25.7	38.5
	23	43.0	64.5	36.2	54.3	28.9	43.3	31.0	46.6	26.2	39.3	23.5	35.2
	24	39.5	59.2	33.3	49.9	26.5	39.8	28.5	42.8	24.1	36.1	21.6	32.3
	25	36.4	54.6	30.7	46.0	24.4	36.7	26.3	39.4	22.2	33.3	19.9	29.8
	26	33.7	50.5	28.3	42.5	22.6	33.9	24.3	36.4	20.5	30.8	18.4	27.5
	28	29.0	43.5	24.4	36.7	19.5	29.2						
	30			21.3	31.9	17.0	25.5						

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	24.8	37.2	19.4	29.2	13.9	20.9	20.1	30.2	15.9	23.8	13.5	20.4
$P_e(KL)^2/10^4$		kip-in.^{2}	747		629		501		540		455		408	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

COMPOSITE
HSS5.000–
HSS4.500

Table 4-18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS5.000×								HSS4.500×			
		0.258		0.250		0.188		0.125		0.375		0.337	
t_{design}, in.		0.240		0.233		0.174		0.116		0.349		0.313	
Steel Wt/ft		13.1		12.7		9.67		6.51		16.5		15.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	113	170	112	167	95.8	144	79.8	120	123	184	114	172
	1	113	170	111	167	95.4	143	79.4	119	122	183	114	171
	2	112	168	110	165	94.3	141	78.4	118	120	180	112	169
	3	110	165	108	162	92.5	139	76.8	115	118	176	110	165
	4	107	160	105	158	90.0	135	74.6	112	114	171	106	159
	5	103	155	102	152	86.9	130	71.8	108	109	164	102	153
	6	99.1	149	97.4	146	83.3	125	68.6	103	104	156	96.9	145
	7	94.3	141	92.8	139	79.2	119	64.9	97.4	97.6	146	91.2	137
	8	89.1	134	87.7	131	74.7	112	60.9	91.4	91.0	137	85.1	128
	9	83.6	125	82.2	123	69.9	105	56.7	85.1	84.1	126	78.7	118
	10	77.8	117	76.5	115	64.9	97.4	52.4	78.5	77.0	116	72.0	108
	11	71.9	108	70.7	106	59.8	89.8	47.9	71.9	69.9	105	65.4	98.1
	12	65.9	98.9	64.8	97.2	54.7	82.1	43.5	65.3	62.8	94.2	58.8	88.1
	13	60.0	90.0	59.0	88.4	49.7	74.5	39.2	58.7	55.9	83.9	52.3	78.5
	14	54.2	81.3	53.2	79.9	44.7	67.1	34.9	52.4	49.3	74.0	46.2	69.3
	15	48.6	72.9	47.7	71.6	39.9	59.9	30.9	46.4	43.1	64.6	40.3	60.5
	16	43.1	64.7	42.4	63.6	35.3	53.0	27.2	40.7	37.8	56.8	35.4	53.2
	17	38.2	57.3	37.5	56.3	31.3	47.0	24.1	36.1	33.5	50.3	31.4	47.1
	18	34.1	51.1	33.5	50.2	27.9	41.9	21.5	32.2	29.9	44.8	28.0	42.0
	19	30.6	45.9	30.1	45.1	25.1	37.6	19.3	28.9	26.8	40.3	25.1	37.7
	20	27.6	41.4	27.1	40.7	22.6	33.9	17.4	26.1	24.2	36.3	22.7	34.0
	21	25.0	37.6	24.6	36.9	20.5	30.8	15.8	23.7	22.0	33.0	20.6	30.9
	22	22.8	34.2	22.4	33.6	18.7	28.0	14.4	21.6	20.0	30.0	18.8	28.1
	23	20.9	31.3	20.5	30.8	17.1	25.7	13.1	19.7	18.3	27.5	17.2	25.7
	24	19.2	28.8	18.8	28.3	15.7	23.6	12.1	18.1	16.8	25.2	15.8	23.6
	25	17.7	26.5	17.4	26.0	14.5	21.7	11.1	16.7				
	26	16.3	24.5	16.1	24.1	13.4	20.1	10.3	15.4				
	28	14.1	21.1	13.8	20.8	11.5	17.3	8.87	13.3				

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	11.4	17.1	11.1	16.7	8.50	12.8	5.80	8.72	12.6	19.0	11.5	17.3
$P_e(KL)^2/10^4$	kip-in.^{2}	363		356		297		229		318		298	

ASD	LRFD	
$\Omega_c = 2.00$	$\phi_c = 0.75$	Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 42 ksi
f'_c = 5 ksi

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

5

COMPOSITE
HSS4.500–
HSS4.000

Shape		HSS4.500×						HSS4.000×			
		0.237		0.188		0.125		0.313		0.250	
t_{design}, in.		0.220		0.174		0.116		0.291		0.233	
Steel Wt/ft		10.8		8.67		5.85		12.3		10.0	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	92.9	139	81.8	123	67.5	101	93.0	139	81.2	122
	1	92.4	139	81.4	122	67.2	101	92.5	139	80.7	121
	2	91.2	137	80.3	120	66.2	99.3	90.8	136	79.3	119
	3	89.1	134	78.4	118	64.5	96.8	88.2	132	77.0	116
	4	86.2	129	75.9	114	62.3	93.4	84.7	127	73.9	111
	5	82.7	124	72.7	109	59.5	89.2	80.3	120	70.1	105
	6	78.5	118	69.0	104	56.2	84.4	75.3	113	65.8	98.6
	7	73.9	111	64.9	97.4	52.6	79.0	69.8	105	60.9	91.4
	8	69.0	103	60.5	90.7	48.8	73.2	63.9	95.8	55.8	83.7
	9	63.7	95.6	55.8	83.7	44.7	67.1	57.8	86.7	50.5	75.8
	10	58.3	87.5	51.0	76.6	40.6	60.9	51.7	77.6	45.2	67.8
	11	52.9	79.3	46.2	69.3	36.5	54.8	45.7	68.6	40.0	60.0
	12	47.5	71.3	41.5	62.2	32.5	48.7	40.0	60.0	34.9	52.4
	13	42.3	63.5	36.9	55.3	28.6	42.9	34.4	51.7	30.1	45.2
	14	37.3	56.0	32.5	48.7	24.9	37.3	29.7	44.5	26.0	38.9
	15	32.6	48.9	28.3	42.4	21.7	32.5	25.9	38.8	22.6	33.9
	16	28.6	42.9	24.9	37.3	19.1	28.6	22.7	34.1	19.9	29.8
	17	25.4	38.0	22.0	33.0	16.9	25.3	20.1	30.2	17.6	26.4
	18	22.6	33.9	19.6	29.5	15.1	22.6	18.0	26.9	15.7	23.6
	19	20.3	30.4	17.6	26.4	13.5	20.3	16.1	24.2	14.1	21.1
	20	18.3	27.5	15.9	23.9	12.2	18.3	14.6	21.8	12.7	19.1
	21	16.6	24.9	14.4	21.7	11.1	16.6	13.2	19.8	11.5	17.3
	22	15.1	22.7	13.2	19.7	10.1	15.1	12.0	18.0	10.5	15.8
	23	13.9	20.8	12.0	18.0	9.22	13.8				
	24	12.7	19.1	11.1	16.6	8.47	12.7				
	25	11.7	17.6	10.2	15.3	7.81	11.7				

Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	8.45	12.7	6.83	10.3	4.67	7.02	8.41	12.6	6.94	10.4
$P_e(KL)^2/10^4$	kip-in.^{2}	240		209		160		192		167	
ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.									
Ω_c = 2.00	ϕ_c = 0.75										

5

COMPOSITE
HSS4.000

Table 4–18 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Round HSS

F_y = 42 ksi
f'_c = 5 ksi

Shape		HSS4.000×									
		0.237		0.226		0.220		0.188		0.125	
t_{design}, in.		0.220		0.210		0.205		0.174		0.116	
Steel Wt/ft		9.53		9.12		8.89		7.66		5.18	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	78.5	118	76.4	115	75.4	113	68.8	103	56.2	84.3
	1	78.0	117	76.0	114	74.9	112	68.4	103	55.9	83.8
	2	76.7	115	74.6	112	73.6	110	67.2	101	54.8	82.2
	3	74.5	112	72.5	109	71.5	107	65.2	97.8	53.1	79.7
	4	71.5	107	69.6	104	68.6	103	62.6	93.9	50.8	76.3
	5	67.8	102	66.0	99.0	65.1	97.6	59.3	89.0	48.0	72.1
	6	63.6	95.3	61.9	92.8	61.0	91.5	55.6	83.4	44.8	67.3
	7	58.9	88.3	57.3	86.0	56.5	84.8	51.5	77.2	41.3	62.0
	8	53.9	80.9	52.5	78.7	51.7	77.6	47.1	70.6	37.6	56.4
	9	48.8	73.2	47.5	71.2	46.8	70.2	42.6	63.9	33.8	50.7
	10	43.7	65.5	42.5	63.7	41.9	62.8	38.1	57.1	30.0	45.0
	11	38.6	57.9	37.5	56.3	37.0	55.5	33.6	50.4	26.3	39.4
	12	33.7	50.6	32.8	49.2	32.3	48.5	29.3	44.0	22.8	34.1
	13	29.1	43.6	28.3	42.4	27.9	41.8	25.2	37.8	19.4	29.2
	14	25.1	37.6	24.4	36.6	24.0	36.0	21.8	32.6	16.8	25.2
	15	21.8	32.8	21.2	31.8	20.9	31.4	19.0	28.4	14.6	21.9
	16	19.2	28.8	18.7	28.0	18.4	27.6	16.7	25.0	12.8	19.3
	17	17.0	25.5	16.5	24.8	16.3	24.4	14.8	22.1	11.4	17.1
	18	15.2	22.7	14.7	22.1	14.5	21.8	13.2	19.7	10.1	15.2
	19	13.6	20.4	13.2	19.8	13.0	19.6	11.8	17.7	9.10	13.7
	20	12.3	18.4	11.9	17.9	11.8	17.7	10.7	16.0	8.22	12.3
	21	11.1	16.7	10.8	16.2	10.7	16.0	9.67	14.5	7.45	11.2
	22	10.2	15.2	9.87	14.8	9.73	14.6	8.81	13.2	6.79	10.2
Properties											
M_n/Ω_b ϕ_bM_n	kip-ft	6.60	9.91	6.33	9.51	6.19	9.31	5.34	8.03	3.67	5.51
$P_e(KL)^2/10^4$	kip-in.^{2}	161		157		154		140		108	

ASD	LRFD
$\Omega_c = 2.00$	$\phi_c = 0.75$

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 35 ksi
f'_c = 4 ksi

Table 4–19
Available Strength in Axial Compression, kips
Concrete Filled Pipe

COMPOSITE PIPE 12–PIPE 8

Shape		Pipe 12				Pipe 10				Pipe 8			
		XS		Std		XS		Std		XXS		XS	
Wall Thickness, in.		0.465		0.349		0.465		0.340		0.816		0.465	
Steel Wt/ft		65.5		49.6		54.8		40.5		72.5		43.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	523	784	455	682	407	610	346	519	423	635	297	445
	6	513	770	446	669	397	595	337	506	407	611	286	429
	7	510	765	443	665	393	590	334	501	401	602	282	423
	8	506	759	440	659	389	584	330	495	395	593	278	416
	9	502	753	436	654	385	577	326	489	388	582	273	409
	10	497	746	431	647	380	570	322	483	380	570	267	401
	11	492	738	427	640	374	561	317	476	372	558	261	392
	12	486	729	421	632	368	553	312	468	363	544	255	383
	13	480	720	416	624	362	543	306	459	353	530	249	373
	14	474	711	410	615	355	533	300	451	343	515	242	362
	15	467	701	404	606	348	523	294	441	333	499	234	351
	16	460	690	397	596	341	511	288	432	322	483	227	340
	17	452	678	390	586	333	500	281	421	311	466	219	328
	18	444	667	383	575	325	488	274	411	299	449	211	317
	19	436	655	376	564	317	476	267	400	288	431	203	304
	20	428	642	368	552	309	463	259	389	276	414	195	292
	21	419	629	360	541	300	450	252	378	264	396	187	280
	22	410	616	352	528	291	437	244	366	252	378	178	267
	23	401	602	344	516	282	424	236	355	240	361	170	255
	24	392	588	336	503	273	410	228	343	229	343	162	243
	25	382	574	327	491	264	397	221	331	217	325	154	231
	26	373	559	318	478	255	383	213	319	205	308	146	218
	27	363	545	310	464	246	369	205	307	194	291	138	207
	28	353	530	301	451	237	355	197	295	183	275	130	195
	29	343	515	292	438	228	342	189	283	172	258	122	184
	30	333	500	283	424	219	328	181	271	161	242	115	172
	32	313	470	265	398	201	301	166	248	142	213	101	151
	34	293	440	247	371	183	275	150	226	126	189	89.5	134
	36	274	410	230	345	166	250	136	204	112	168	79.8	120
	38	254	381	212	319	150	225	122	183	101	151	71.6	107
	40	235	353	196	294	135	203	110	165	90.8	136	64.6	97.0

Properties

M_n/Ω_b	$\phi_b M_n$ kip-ft	123	184	93.8	141	86.0	129	64.4	96.8	87.2	131	54.1	81.4
$P_e(KL)^2/10^4$	kip-in.^{2}	12600		10400		7110		5790		4770		3400	
ASD	LRFD												
Ω_c = 2.00	ϕ_c = 0.75												

4

COMPOSITE
PIPE 8–PIPE 5

Table 4–19 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Pipe

F_y = 35 ksi
f'_c = 4 ksi

Shape		Pipe 8		Pipe 6						Pipe 5	
		Std		XXS		XS		Std		XXS	
Wall Thickness, in.		0.300		0.805		0.403		0.261		0.699	
Steel Wt/ft		28.6		53.2		28.6		19.0		38.6	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	233	350	295	442	188	283	147	220	213	320
	6	224	337	276	414	177	265	138	206	194	291
	7	221	332	269	404	173	259	134	202	187	281
	8	218	326	262	393	168	252	131	196	180	270
	9	214	320	254	380	163	245	127	190	172	258
	10	209	314	245	367	158	236	122	184	163	245
	11	204	307	235	353	152	228	118	177	155	232
	12	199	299	226	338	146	219	113	169	145	218
	13	194	291	215	323	139	209	108	162	136	204
	14	188	282	205	307	133	199	103	154	127	190
	15	182	273	194	291	126	189	97.4	146	117	176
	16	176	264	183	275	119	179	92.0	138	108	162
	17	170	255	172	259	112	169	86.6	130	98.9	148
	18	163	245	162	242	106	158	81.2	122	90.2	135
	19	157	235	151	226	98.8	148	75.9	114	81.6	122
	20	150	226	140	210	92.2	138	70.7	106	73.7	110
	21	144	216	130	195	85.7	129	65.6	98.4	66.8	100
	22	137	206	120	180	79.3	119	60.6	91.0	60.9	91.3
	23	130	196	110	165	73.2	110	55.8	83.6	55.7	83.5
	24	124	186	101	152	67.2	101	51.2	76.8	51.1	76.7
	25	117	176	93.2	140	61.9	92.9	47.2	70.8	47.1	70.7
	26	111	166	86.2	129	57.2	85.9	43.6	65.4	43.6	65.4
	27	105	157	79.9	120	53.1	79.6	40.5	60.7	40.4	60.6
	28	98.5	148	74.3	112	49.4	74.0	37.6	56.4	37.6	56.4
	29	92.5	139	69.3	104	46.0	69.0	35.1	52.6	35.0	52.5
	30	86.5	130	64.8	97.1	43.0	64.5	32.8	49.2		
	32	76.1	114	56.9	85.4	37.8	56.7	28.8	43.2		
	34	67.4	101	50.4	75.6	33.5	50.2	25.5	38.3		
	36	60.1	90.1			29.9	44.8	22.8	34.1		
	38	53.9	80.9								
	40	48.7	73.0								

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	36.3	54.6	47.9	72.0	27.3	41.0	18.5	27.8	29.1	43.7
$P_e(KL)^2/10^4$		kip-in.^{2}	2560		1920		1270		969		968	
ASD	**LRFD**		Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$											

F_y = 35 ksi
f'_c = 4 ksi

Table 4–19 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Pipe

4

COMPOSITE PIPE 5–PIPE 4

Shape		Pipe 5				Pipe 4					
		XS		Std		XXS		XS		Std	
Wall Thickness, in.		0.349		0.241		0.628		0.315		0.221	
Steel Wt/ft		20.8		14.6		27.6		15.0		10.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	135	203	109	164	149	224	94.8	142	76.6	115
	6	124	185	99.4	149	129	193	82.5	124	66.5	99.8
	7	120	179	96.1	144	122	183	78.4	118	63.2	94.8
	8	115	173	92.5	139	115	172	74.0	111	59.6	89.4
	9	110	165	88.5	133	107	161	69.3	104	55.8	83.7
	10	105	158	84.3	126	99.0	148	64.4	96.6	51.8	77.7
	11	99.6	149	79.9	120	90.8	136	59.3	89.0	47.7	71.6
	12	94.0	141	75.3	113	82.6	124	54.3	81.4	43.6	65.5
	13	88.2	132	70.6	106	74.5	112	49.3	73.9	39.6	59.4
	14	82.4	124	65.8	98.8	66.7	100	44.4	66.6	35.6	53.4
	15	76.5	115	61.1	91.7	59.2	88.8	39.7	59.5	31.8	47.7
	16	70.7	106	56.4	84.6	52.1	78.1	35.1	52.7	28.1	42.2
	17	65.1	97.6	51.8	77.7	46.1	69.2	31.1	46.7	24.9	37.4
	18	59.5	89.3	47.4	71.0	41.1	61.7	27.7	41.6	22.2	33.3
	19	54.2	81.3	43.1	64.6	36.9	55.4	24.9	37.3	19.9	29.9
	20	49.0	73.5	38.9	58.3	33.3	50.0	22.5	33.7	18.0	27.0
	21	44.4	66.6	35.3	52.9	30.2	45.3	20.4	30.6	16.3	24.5
	22	40.5	60.7	32.1	48.2	27.5	41.3	18.6	27.9	14.9	22.3
	23	37.0	55.6	29.4	44.1	25.2	37.8	17.0	25.5	13.6	20.4
	24	34.0	51.0	27.0	40.5			15.6	23.4	12.5	18.7
	25	31.3	47.0	24.9	37.3					11.5	17.3
	26	29.0	43.5	23.0	34.5						
	27	26.9	40.3	21.3	32.0						
	28	25.0	37.5	19.8	29.8						
	29	23.3	34.9	18.5	27.7						
	30	21.8	32.7	17.3	25.9						

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	16.6	24.9	11.9	17.9	16.6	24.9	9.65	14.5	7.07	10.6
$P_e(KL)^2/10^4$		kip-in.^{2}	644		511		438		295		236	

ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.
$\Omega_c = 2.00$	$\phi_c = 0.75$	

COMPOSITE
PIPE $3^1/_2$–PIPE 3

Table 4–19 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Pipe

F_y = 35 ksi
f'_c = 4 ksi

Shape		Pipe $3^1/_2$				Pipe 3					
		XS		Std		XXS		XS		Std	
Wall Thickness, in.		0.296		0.211		0.559		0.280		0.201	
Steel Wt/ft		12.5		9.12		18.6		10.3		7.58	
Design		P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD	P_n/Ω_c ASD	$\phi_c P_n$ LRFD
Effective length (KL) (ft)	0	77.5	116	63.0	94.5	98.8	148	62.4	93.6	50.7	76.1
	6	65.0	97.5	52.8	79.2	76.8	115	49.6	74.3	40.3	60.4
	7	61.0	91.5	49.5	74.3	70.2	105	45.6	68.4	37.1	55.6
	8	56.6	85.0	46.0	69.0	63.2	94.8	41.4	62.1	33.7	50.5
	9	52.1	78.2	42.3	63.5	56.1	84.2	37.1	55.7	30.2	45.3
	10	47.5	71.2	38.5	57.8	49.2	73.7	32.9	49.3	26.7	40.1
	11	42.8	64.2	34.7	52.1	42.5	63.7	28.7	43.1	23.4	35.1
	12	38.3	57.4	31.0	46.5	36.1	54.1	24.8	37.2	20.2	30.2
	13	33.8	50.8	27.4	41.1	30.7	46.1	21.1	31.7	17.2	25.8
	14	29.6	44.4	24.0	36.0	26.5	39.8	18.2	27.3	14.8	22.2
	15	25.8	38.7	20.9	31.3	23.1	34.6	15.9	23.8	12.9	19.4
	16	22.7	34.0	18.4	27.5	20.3	30.4	14.0	20.9	11.4	17.0
	17	20.1	30.1	16.3	24.4	18.0	27.0	12.4	18.5	10.1	15.1
	18	17.9	26.9	14.5	21.8			11.0	16.5	8.97	13.5
	19	16.1	24.1	13.0	19.5			9.89	14.8	8.05	12.1
	20	14.5	21.8	11.8	17.6						
	21	13.2	19.7	10.7	16.0						
	22			9.71	14.6						

Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	7.11	10.7	5.30	7.96	8.55	12.8	5.08	7.64	3.83	5.75
$P_e(KL)^2/10^4$	kip-in.^{2}	190		154		170		117		95.5	
ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 35 ksi
f'_c = 5 ksi

Table 4–20
Available Strength in Axial Compression, kips
Concrete Filled Pipe

⑤
COMPOSITE
PIPE 12–PIPE 8

Shape		Pipe 12				Pipe 10				Pipe 8			
		XS		Std		XS		Std		XXS		XS	
Wall Thickness, in.		0.465		0.349		0.465		0.340		0.816		0.465	
Steel Wt/ft		65.5		49.6		54.8		40.5		72.5		43.4	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	575	862	509	763	443	664	384	576	441	662	319	479
	6	564	846	499	748	431	647	373	560	424	636	307	460
	7	560	840	495	742	427	641	369	554	418	627	302	453
	8	555	833	491	736	423	634	365	548	411	617	297	446
	9	550	825	486	729	417	626	360	541	403	605	292	437
	10	545	817	481	721	412	617	355	533	395	593	285	428
	11	539	808	475	712	405	608	349	524	386	579	279	418
	12	532	798	469	703	398	598	343	515	376	565	272	408
	13	525	788	462	693	391	587	337	505	366	549	264	397
	14	517	776	455	683	384	575	330	494	355	533	257	385
	15	510	764	448	671	375	563	322	483	344	516	248	373
	16	501	752	440	660	367	551	315	472	333	499	240	360
	17	492	739	431	647	358	537	307	460	321	481	231	347
	18	483	725	423	634	349	524	298	448	308	463	223	334
	19	474	711	414	621	340	510	290	435	296	444	214	320
	20	464	696	405	607	330	495	281	422	284	425	205	307
	21	454	681	396	593	320	481	273	409	271	407	195	293
	22	444	665	386	579	311	466	264	395	259	388	186	279
	23	433	650	376	564	300	451	255	382	246	369	177	266
	24	422	634	366	549	290	435	246	368	234	350	168	252
	25	411	617	356	534	280	420	236	355	221	332	159	239
	26	400	601	346	519	270	405	227	341	209	314	150	226
	27	389	584	336	503	259	389	218	327	197	296	142	213
	28	378	567	325	488	249	374	209	313	186	278	133	200
	29	367	550	315	472	239	358	200	300	174	261	125	188
	30	355	533	304	457	229	343	191	286	163	244	117	176
	32	332	499	284	425	209	313	173	260	143	215	103	154
	34	310	465	263	395	190	285	157	235	127	190	91.1	137
	36	287	431	243	364	171	257	140	210	113	170	81.3	122
	38	266	398	223	335	153	230	126	189	102	152	72.9	109
	40	244	366	204	306	139	208	114	170	91.6	137	65.8	98.7
Properties													
M_n/Ω_b	$\phi_b M_n$ kip-ft	123	184	93.8	141	86.0	129	64.4	96.8	87.2	131	54.1	81.4
$P_e(KL)^2/10^4$	kip-in.^{2}	13000		10700		7270		5960		4810		3460	
ASD	**LRFD**												
$\Omega_c = 2.00$	$\phi_c = 0.75$												

5

COMPOSITE
PIPE 8–PIPE 5

Table 4–20 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Pipe

F_y = 35 ksi
f'_c = 5 ksi

Shape		Pipe 8		Pipe 6						Pipe 5	
		Std		XXS		XS		Std		XXS	
Wall Thickness, in.		0.300		0.805		0.403		0.261		0.699	
Steel Wt/ft		28.6		53.2		28.6		19.0		38.6	
Design		P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n	P_n/Ω_c	ϕ_cP_n
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (KL) (ft)	0	257	386	304	456	201	302	161	241	220	329
	6	247	370	284	426	188	282	150	225	199	299
	7	243	365	277	416	183	275	146	219	192	288
	8	239	358	269	404	178	267	142	213	184	277
	9	234	351	261	391	173	259	137	206	176	264
	10	229	343	251	377	167	250	132	198	167	251
	11	223	335	242	362	160	240	127	191	158	237
	12	217	326	231	347	153	230	121	182	148	223
	13	211	316	220	331	146	220	116	173	139	208
	14	204	306	209	314	139	209	110	165	129	193
	15	197	296	198	297	132	198	104	156	119	179
	16	190	286	187	280	124	187	97.6	146	109	164
	17	183	275	175	263	117	175	91.5	137	100	150
	18	176	263	164	246	109	164	85.5	128	90.9	136
	19	168	252	153	229	102	153	79.5	119	82.1	123
	20	161	241	142	213	94.9	142	73.7	111	74.1	111
	21	153	230	131	197	87.9	132	68.0	102	67.2	101
	22	145	218	121	182	81.1	122	62.5	93.8	61.2	91.8
	23	138	207	111	166	74.3	112	57.2	85.8	56.0	84.0
	24	130	196	102	153	68.3	102	52.5	78.8	51.4	77.1
	25	123	185	93.8	141	62.9	94.4	48.4	72.6	47.4	71.1
	26	116	174	86.7	130	58.2	87.3	44.8	67.1	43.8	65.7
	27	109	163	80.4	121	53.9	80.9	41.5	62.3	40.6	61.0
	28	102	153	74.8	112	50.2	75.2	38.6	57.9	37.8	56.7
	29	95.2	143	69.7	105	46.8	70.1	36.0	54.0	35.2	52.8
	30	89.0	133	65.1	97.7	43.7	65.5	33.6	50.4		
	32	78.2	117	57.3	85.9	38.4	57.6	29.5	44.3		
	34	69.3	104	50.7	76.1	34.0	51.0	26.2	39.3		
	36	61.8	92.7			30.3	45.5	23.3	35.0		
	38	55.5	83.2								
	40	50.0	75.1								
Properties											
M_n/Ω_b	ϕ_bM_n kip-ft	36.3	54.6	47.9	72.0	27.3	41.0	18.5	27.8	29.1	43.7
$P_e(KL)^2/10^4$	kip-in.^{2}	2640		1930		1290		994		973	
ASD	LRFD	Note: Heavy line indicates Kl/r equal to or greater than 200.									
$\Omega_c = 2.00$	$\phi_c = 0.75$										

F_y = 35 ksi
f'_c = 5 ksi

Table 4-20 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Pipe

5

COMPOSITE PIPE 5–PIPE 4

Shape		Pipe 5				Pipe 4					
		XS		Std		XXS		XS		Std	
Wall Thickness, in.		0.349		0.241		0.628		0.315		0.221	
Steel Wt/ft		20.8		14.6		27.6		15.0		10.8	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length (*KL*) (ft)	**0**	144	216	119	178	153	230	100	151	82.7	124
	6	131	197	108	161	132	198	86.8	130	71.3	107
	7	127	190	104	156	125	187	82.3	124	67.5	101
	8	122	182	99.6	149	117	176	77.5	116	63.5	95.2
	9	116	174	95.1	143	109	164	72.3	109	59.2	88.8
	10	111	166	90.3	135	101	151	67.0	100	54.7	82.1
	11	105	157	85.2	128	92.2	138	61.5	92.3	50.2	75.2
	12	98.3	147	80.0	120	83.7	126	56.1	84.1	45.6	68.4
	13	92.0	138	74.7	112	75.3	113	50.7	76.0	41.1	61.7
	14	85.6	128	69.4	104	67.3	101	45.4	68.2	36.8	55.2
	15	79.3	119	64.1	96.2	59.5	89.2	40.4	60.6	32.6	49.0
	16	73.0	109	58.9	88.3	52.3	78.4	35.6	53.4	28.7	43.1
	17	66.9	100	53.8	80.7	46.3	69.5	31.5	47.3	25.4	38.1
	18	60.9	91.4	48.9	73.3	41.3	62.0	28.1	42.2	22.7	34.0
	19	55.1	82.7	44.1	66.1	37.1	55.6	25.2	37.9	20.4	30.5
	20	49.7	74.6	39.8	59.7	33.5	50.2	22.8	34.2	18.4	27.6
	21	45.1	67.7	36.1	54.1	30.4	45.5	20.7	31.0	16.7	25.0
	22	41.1	61.7	32.9	49.3	27.7	41.5	18.8	28.2	15.2	22.8
	23	37.6	56.4	30.1	45.1	25.3	38.0	17.2	25.8	13.9	20.8
	24	34.5	51.8	27.6	41.4			15.8	23.7	12.8	19.1
	25	31.8	47.8	25.5	38.2					11.8	17.6
	26	29.4	44.2	23.5	35.3						
	27	27.3	40.9	21.8	32.7						
	28	25.4	38.1	20.3	30.4						
	29	23.7	35.5	18.9	28.4						
	30	22.1	33.2	17.7	26.5						

Properties

M_n/Ω_b	$\phi_b M_n$	kip-ft	16.6	24.9	11.9	17.9	16.6	24.9	9.65	14.5	7.07	10.6
$P_e(KL)^2/10^4$		kip-in.^{2}	654		523		439		300		242	

ASD	**LRFD**	Note: Heavy line indicates *Kl/r* equal to or greater than 200.
$\Omega_c = 2.00$	$\phi_c = 0.75$	

5

COMPOSITE
PIPE 3$^1/_2$–PIPE 3

Table 4–20 (continued)
Available Strength in Axial Compression, kips
Concrete Filled Pipe

F_y = 35 ksi
f'_c = 5 ksi

Shape		Pipe 3$^1/_2$				Pipe 3					
		XS		Std		XXS		XS		Std	
Wall Thickness, in.		0.296		0.211		0.559		0.280		0.201	
Steel Wt/ft		12.5		9.12		18.6		10.3		7.58	
Design		P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$	P_n/Ω_c	$\phi_c P_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length (*KL*) (ft)	0	81.9	123	67.8	102	101	151	65.7	98.5	54.3	81.5
	6	68.1	102	56.2	84.4	78.1	117	51.6	77.5	42.6	63.9
	7	63.7	95.6	52.6	78.8	71.2	107	47.3	71.0	39.0	58.5
	8	59.0	88.5	48.6	72.9	64.0	96.1	42.8	64.3	35.3	52.9
	9	54.1	81.2	44.5	66.8	56.8	85.1	38.2	57.4	31.5	47.2
	10	49.1	73.6	40.3	60.5	49.6	74.4	33.7	50.5	27.7	41.5
	11	44.1	66.1	36.2	54.2	42.7	64.1	29.3	43.9	24.0	36.0
	12	39.2	58.8	32.1	48.1	36.2	54.3	25.1	37.6	20.5	30.8
	13	34.5	51.7	28.2	42.3	30.8	46.3	21.4	32.1	17.5	26.2
	14	30.0	45.0	24.4	36.7	26.6	39.9	18.4	27.6	15.1	22.6
	15	26.1	39.2	21.3	31.9	23.2	34.8	16.1	24.1	13.1	19.7
	16	23.0	34.4	18.7	28.1	20.4	30.5	14.1	21.2	11.6	17.3
	17	20.3	30.5	16.6	24.9	18.0	27.1	12.5	18.7	10.2	15.3
	18	18.1	27.2	14.8	22.2			11.1	16.7	9.13	13.7
	19	16.3	24.4	13.3	19.9			10.0	15.0	8.19	12.3
	20	14.7	22.0	12.0	18.0						
	21	13.3	20.0	10.9	16.3						
	22			9.90	14.8						
Properties											
M_n/Ω_b	$\phi_b M_n$ kip-ft	7.11	10.7	5.30	7.96	8.55	12.8	5.08	7.64	3.83	5.75
$P_e(KL)^2/10^4$	kip-in.^{2}	193		157		171		119		97.1	
ASD	LRFD	Note: Heavy line indicates *Kl/r* equal to or greater than 200.									
Ω_c = 2.00	ϕ_c = 0.75										

Table 4–21
Stiffness Reduction Factor τ_a

ASD	LRFD	F_y, ksi									
$\frac{P_a}{A_g}$	$\frac{P_u}{A_g}$	35		36		42		46		50	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
45		–	–	–	–	–	–	–	–	–	–
44		–	–	–	–	–	–	–	–	–	0.0599
43		–	–	–	–	–	–	–	–	–	0.118
42		–	–	–	–	–	–	–	–	–	0.175
41		–	–	–	–	–	–	–	0.0262	–	0.231
40		–	–	–	–	–	–	–	0.0905	–	0.285
39		–	–	–	–	–	–	–	0.153	–	0.338
38		–	–	–	–	–	–	–	0.214	–	0.389
37		–	–	–	–	–	0.0570	–	0.274	–	0.438
36		–	–	–	–	–	0.127	–	0.331	–	0.486
35		–	–	–	–	–	0.194	–	0.387	–	0.532
34		–	–	–	–	–	0.260	–	0.441	–	0.577
33		–	–	–	–	–	0.323	–	0.492	–	0.620
32		–	–	–	0.0334	–	0.384	–	0.542	–	0.660
31		–	0.0429	–	0.115	–	0.443	–	0.590	–	0.699
30		–	0.127	–	0.194	–	0.500	–	0.636	–	0.736
29		–	0.207	–	0.270	–	0.554	–	0.679	0.0842	0.771
28		–	0.285	–	0.344	–	0.606	–	0.720	0.171	0.804
27		–	0.360	–	0.414	–	0.655	0.0534	0.759	0.254	0.835
26		–	0.431	–	0.481	–	0.701	0.148	0.796	0.334	0.863
25		–	0.500	–	0.545	0.0162	0.745	0.240	0.830	0.410	0.890
24		–	0.564	–	0.606	0.122	0.786	0.327	0.861	0.483	0.913
23		–	0.626	–	0.663	0.223	0.823	0.410	0.890	0.552	0.934
22		–	0.683	–	0.716	0.319	0.858	0.489	0.915	0.617	0.953
21		–	0.736	0.0695	0.766	0.410	0.890	0.563	0.938	0.678	0.969
20		0.122	0.786	0.189	0.811	0.496	0.917	0.633	0.957	0.734	0.982
19		0.242	0.831	0.303	0.853	0.577	0.942	0.698	0.974	0.786	0.992
18		0.356	0.871	0.410	0.890	0.652	0.962	0.757	0.986	0.833	0.998
17		0.462	0.907	0.510	0.922	0.721	0.979	0.811	0.996	0.875	1.00
16		0.561	0.937	0.603	0.949	0.784	0.991	0.860	1.00	0.912	↓
15		0.652	0.962	0.687	0.971	0.840	0.999	0.902	↓	0.943	↓
14		0.734	0.982	0.764	0.988	0.888	1.00	0.937	↓	0.968	↓
13		0.807	0.995	0.831	0.998	0.929	↓	0.965	↓	0.987	↓
12		0.870	1.00	0.888	1.00	0.962	↓	0.986	↓	0.998	↓
11		0.922	↓	0.935	↓	0.985	↓	0.999	↓	1.00	↓
10		0.962	↓	0.971	↓	0.999	↓	1.00	↓	↓	↓
9		0.989	↓	0.993	↓	1.00	↓	↓	↓	↓	↓
8		1.00	↓	1.00	↓	↓	↓	↓	↓	↓	↓
7		↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
6		↓	↓	↓	↓	↓	↓	↓	↓	↓	↓
5		↓	↓	↓	↓	↓	↓	↓	↓	↓	↓

– Indicates stiffness reduction factor is not applicable because the required strength exceeds the available strength for $Kl/r = 0$.

Table 4–22
Available Critical Stress for Compression Members

$F_y = 35$ksi			$F_y = 36$ksi			$F_y = 42$ksi			$F_y = 46$ksi			$F_y = 50$ksi		
$\frac{Kl}{r}$	F_{cr}/Ω_c ksi **ASD**	$\phi_c F_{cr}$ ksi **LRFD**	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi **ASD**	$\phi_c F_{cr}$ ksi **LRFD**	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi **ASD**	$\phi_c F_{cr}$ ksi **LRFD**	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi **ASD**	$\phi_c F_{cr}$ ksi **LRFD**	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi **ASD**	$\phi_c F_{cr}$ ksi **LRFD**
1	21.0	31.5	1	21.6	32.4	1	25.1	37.8	1	27.5	41.4	1	29.9	45.0
2	21.0	31.5	2	21.6	32.4	2	25.1	37.8	2	27.5	41.4	2	29.9	45.0
3	20.9	31.5	3	21.5	32.4	3	25.1	37.8	3	27.5	41.4	3	29.9	45.0
4	20.9	31.5	4	21.5	32.4	4	25.1	37.8	4	27.5	41.4	4	29.9	44.9
5	20.9	31.5	5	21.5	32.4	5	25.1	37.7	5	27.5	41.3	5	29.9	44.9
6	20.9	31.4	6	21.5	32.3	6	25.1	37.7	6	27.5	41.3	6	29.9	44.9
7	20.9	31.4	7	21.5	32.3	7	25.1	37.7	7	27.5	41.3	7	29.8	44.8
8	20.9	31.4	8	21.5	32.3	8	25.1	37.7	8	27.4	41.2	8	29.8	44.8
9	20.9	31.4	9	21.5	32.3	9	25.0	37.6	9	27.4	41.2	9	29.8	44.7
10	20.9	31.3	10	21.4	32.2	10	25.0	37.6	10	27.4	41.1	10	29.7	44.7
11	20.8	31.3	11	21.4	32.2	11	25.0	37.5	11	27.3	41.1	11	29.7	44.6
12	20.8	31.3	12	21.4	32.2	12	24.9	37.5	12	27.3	41.0	12	29.6	44.5
13	20.8	31.2	13	21.4	32.1	13	24.9	37.4	13	27.2	40.9	13	29.6	44.4
14	20.7	31.2	14	21.3	32.1	14	24.8	37.3	14	27.2	40.9	14	29.5	44.4
15	20.7	31.1	15	21.3	32.0	15	24.8	37.3	15	27.1	40.8	15	29.5	44.3
16	20.7	31.1	16	21.3	32.0	16	24.8	37.2	16	27.1	40.7	16	29.4	44.2
17	20.7	31.0	17	21.2	31.9	17	24.7	37.1	17	27.0	40.6	17	29.3	44.1
18	20.6	31.0	18	21.2	31.9	18	24.7	37.1	18	27.0	40.5	18	29.2	43.9
19	20.6	30.9	19	21.2	31.8	19	24.6	37.0	19	26.9	40.4	19	29.2	43.8
20	20.5	30.9	20	21.1	31.7	20	24.5	36.9	20	26.8	40.3	20	29.1	43.7
21	20.5	30.8	21	21.1	31.7	21	24.5	36.8	21	26.7	40.2	21	29.0	43.6
22	20.4	30.7	22	21.0	31.6	22	24.4	36.7	22	26.7	40.1	22	28.9	43.4
23	20.4	30.7	23	21.0	31.5	23	24.3	36.6	23	26.6	40.0	23	28.8	43.3
24	20.3	30.6	24	20.9	31.4	24	24.3	36.5	24	26.5	39.8	24	28.7	43.1
25	20.3	30.5	25	20.9	31.4	25	24.2	36.4	25	26.4	39.7	25	28.6	43.0
26	20.2	30.4	26	20.8	31.3	26	24.1	36.3	26	26.3	39.6	26	28.5	42.8
27	20.2	30.3	27	20.7	31.2	27	24.0	36.1	27	26.2	39.4	27	28.4	42.7
28	20.1	30.3	28	20.7	31.1	28	24.0	36.0	28	26.1	39.3	28	28.3	42.5
29	20.1	30.2	29	20.6	31.0	29	23.9	35.9	29	26.0	39.1	29	28.2	42.3
30	20.0	30.1	30	20.6	30.9	30	23.8	35.8	30	25.9	39.0	30	28.0	42.1
31	20.0	30.0	31	20.5	30.8	31	23.7	35.6	31	25.8	38.8	31	27.9	41.9
32	19.9	29.9	32	20.4	30.7	32	23.6	35.5	32	25.7	38.6	32	27.8	41.8
33	19.8	29.8	33	20.4	30.6	33	23.5	35.4	33	25.6	38.5	33	27.7	41.6
34	19.8	29.7	34	20.3	30.5	34	23.4	35.2	34	25.5	38.3	34	27.5	41.4
35	19.7	29.6	35	20.2	30.4	35	23.3	35.1	35	25.4	38.1	35	27.4	41.2
36	19.6	29.5	36	20.1	30.3	36	23.2	34.9	36	25.2	37.9	36	27.2	40.9
37	19.5	29.4	37	20.1	30.1	37	23.1	34.8	37	25.1	37.8	37	27.1	40.7
38	19.5	29.3	38	20.0	30.0	38	23.0	34.6	38	25.0	37.6	38	26.9	40.5
39	19.4	29.1	39	19.9	29.9	39	22.9	34.4	39	24.9	37.4	39	26.8	40.3
40	19.3	29.0	40	19.8	29.8	40	22.8	34.3	40	24.7	37.2	40	26.6	40.0

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–22 (continued)

Available Critical Stress for Compression Members

	F_y = 35ksi			F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi	
$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	*ksi*		*ksi*	*ksi*		*ksi*	*ksi*		*ksi*	*ksi*		*ksi*	*ksi*
	ASD	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**
41	**19.2**	28.9	41	**19.7**	29.7	41	**22.7**	34.1	41	**24.6**	37.0	41	**26.5**	39.8
42	**19.2**	28.8	42	**19.6**	29.5	42	**22.6**	33.9	42	**24.5**	36.8	42	**26.3**	39.5
43	**19.1**	28.7	43	**19.6**	29.4	43	**22.5**	33.7	43	**24.3**	36.6	43	**26.2**	39.3
44	**19.0**	28.5	44	**19.5**	29.3	44	**22.3**	33.6	44	**24.2**	36.3	44	**26.0**	39.1
45	**18.9**	28.4	45	**19.4**	29.1	45	**22.2**	33.4	45	**24.0**	36.1	45	**25.8**	38.8
46	**18.8**	28.3	46	**19.3**	29.0	46	**22.1**	33.2	46	**23.9**	35.9	46	**25.6**	38.5
47	**18.7**	28.1	47	**19.2**	28.9	47	**22.0**	33.0	47	**23.8**	35.7	47	**25.5**	38.3
48	**18.6**	28.0	48	**19.1**	28.7	48	**21.8**	32.8	48	**23.6**	35.4	48	**25.3**	38.0
49	**18.5**	27.9	49	**19.0**	28.5	49	**21.7**	32.6	49	**23.4**	35.2	49	**25.1**	37.7
50	**18.4**	27.7	50	**18.9**	28.4	50	**21.6**	32.4	50	**23.3**	35.0	50	**24.9**	37.5
51	**18.3**	27.6	51	**18.8**	28.3	51	**21.4**	32.2	51	**23.1**	34.8	51	**24.8**	37.2
52	**18.3**	27.4	52	**18.7**	28.1	52	**21.3**	32.0	52	**23.0**	34.5	52	**24.6**	36.9
53	**18.2**	27.3	53	**18.6**	28.0	53	**21.2**	31.8	53	**22.8**	34.3	53	**24.4**	36.7
54	**18.1**	27.1	54	**18.5**	27.8	54	**21.0**	31.6	54	**22.6**	34.0	54	**24.2**	36.4
55	**18.0**	27.0	55	**18.4**	27.6	55	**20.9**	31.4	55	**22.5**	33.8	55	**24.0**	36.1
56	**17.9**	26.8	56	**18.3**	27.5	56	**20.7**	31.2	56	**22.3**	33.5	56	**23.8**	35.8
57	**17.7**	26.7	57	**18.2**	27.3	57	**20.6**	31.0	57	**22.1**	33.3	57	**23.6**	35.5
58	**17.6**	26.5	58	**18.1**	27.1	58	**20.5**	30.7	58	**22.0**	33.0	58	**23.4**	35.2
59	**17.5**	26.4	59	**17.9**	27.0	59	**20.3**	30.5	59	**21.8**	32.8	59	**23.2**	34.9
60	**17.4**	26.2	60	**17.8**	26.8	60	**20.2**	30.3	60	**21.6**	32.5	60	**23.0**	34.6
61	**17.3**	26.0	61	**17.7**	26.6	61	**20.0**	30.1	61	**21.4**	32.2	61	**22.8**	34.3
62	**17.2**	25.9	62	**17.6**	26.5	62	**19.9**	29.9	62	**21.3**	32.0	62	**22.6**	34.0
63	**17.1**	25.7	63	**17.5**	26.3	63	**19.7**	29.6	63	**21.1**	31.7	63	**22.4**	33.7
64	**17.0**	25.5	64	**17.4**	26.1	64	**19.6**	29.4	64	**20.9**	31.4	64	**22.2**	33.4
65	**16.9**	25.4	65	**17.3**	25.9	65	**19.4**	29.2	65	**20.7**	31.2	65	**22.0**	33.0
66	**16.8**	25.2	66	**17.1**	25.8	66	**19.2**	28.9	66	**20.5**	30.9	66	**21.8**	32.7
67	**16.7**	25.0	67	**17.0**	25.6	67	**19.1**	28.7	67	**20.4**	30.6	67	**21.6**	32.4
68	**16.5**	24.9	68	**16.9**	25.4	68	**18.9**	28.5	68	**20.2**	30.3	68	**21.4**	32.1
69	**16.4**	24.7	69	**16.8**	25.2	69	**18.8**	28.2	69	**20.0**	30.1	69	**21.1**	31.8
70	**16.3**	24.5	70	**16.7**	25.0	70	**18.6**	28.0	70	**19.8**	29.8	70	**20.9**	31.4
71	**16.2**	24.3	71	**16.5**	24.8	71	**18.5**	27.7	71	**19.6**	29.5	71	**20.7**	31.1
72	**16.1**	24.2	72	**16.4**	24.7	72	**18.3**	27.5	72	**19.4**	29.2	72	**20.5**	30.8
73	**16.0**	24.0	73	**16.3**	24.5	73	**18.1**	27.2	73	**19.2**	28.9	73	**20.3**	30.5
74	**15.8**	23.8	74	**16.2**	24.3	74	**18.0**	27.0	74	**19.1**	28.6	74	**20.1**	30.2
75	**15.7**	23.6	75	**16.0**	24.1	75	**17.8**	26.8	75	**18.9**	28.4	75	**19.8**	29.8
76	**15.6**	23.4	76	**15.9**	23.9	76	**17.6**	26.5	76	**18.7**	28.1	76	**19.6**	29.5
77	**15.5**	23.3	77	**15.8**	23.7	77	**17.5**	26.3	77	**18.5**	27.8	77	**19.4**	29.2
78	**15.4**	23.1	78	**15.6**	23.5	78	**17.3**	26.0	78	**18.3**	27.5	78	**19.2**	28.8
79	**15.2**	22.9	79	**15.5**	23.3	79	**17.1**	25.8	79	**18.1**	27.2	79	**19.0**	28.5
80	**15.1**	22.7	80	**15.4**	23.1	80	**17.0**	25.5	80	**17.9**	26.9	80	**18.8**	28.2

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–22 (continued)
Available Critical Stress for Compression Members

F_y = 35ksi			F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi		
$\frac{Kl}{r}$	F_{cr}/Ω_c ksi	$\phi_c F_{cr}$ ksi	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi	$\phi_c F_{cr}$ ksi	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi	$\phi_c F_{cr}$ ksi	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi	$\phi_c F_{cr}$ ksi	$\frac{Kl}{r}$	F_{cr}/Ω_c ksi	$\phi_c F_{cr}$ ksi
	ASD	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**
81	**15.0**	22.5	81	**15.3**	22.9	81	**16.8**	25.3	81	**17.7**	26.6	81	**18.5**	27.9
82	**14.9**	22.3	82	**15.1**	22.7	82	**16.6**	25.0	82	**17.5**	26.3	82	**18.3**	27.5
83	**14.7**	22.1	83	**15.0**	22.5	83	**16.5**	24.8	83	**17.3**	26.0	83	**18.1**	27.2
84	**14.6**	22.0	84	**14.9**	22.3	84	**16.3**	24.5	84	**17.1**	25.8	84	**17.9**	26.9
85	**14.5**	21.8	85	**14.7**	22.1	85	**16.1**	24.3	85	**16.9**	25.5	85	**17.7**	26.5
86	**14.4**	21.6	86	**14.6**	22.0	86	**16.0**	24.0	86	**16.7**	25.2	86	**17.4**	26.2
87	**14.2**	21.4	87	**14.5**	21.8	87	**15.8**	23.7	87	**16.6**	24.9	87	**17.2**	25.9
88	**14.1**	21.2	88	**14.3**	21.6	88	**15.6**	23.5	88	**16.4**	24.6	88	**17.0**	25.5
89	**14.0**	21.0	89	**14.2**	21.4	89	**15.5**	23.2	89	**16.2**	24.3	89	**16.8**	25.2
90	**13.8**	20.8	90	**14.1**	21.2	90	**15.3**	23.0	90	**16.0**	24.0	90	**16.6**	24.9
91	**13.7**	20.6	91	**13.9**	21.0	91	**15.1**	22.7	91	**15.8**	23.7	91	**16.3**	24.6
92	**13.6**	20.4	92	**13.8**	20.8	92	**15.0**	22.5	92	**15.6**	23.4	92	**16.1**	24.2
93	**13.5**	20.2	93	**13.7**	20.5	93	**14.8**	22.2	93	**15.4**	23.1	93	**15.9**	23.9
94	**13.3**	20.0	94	**13.5**	20.3	94	**14.6**	22.0	94	**15.2**	22.8	94	**15.7**	23.6
95	**13.2**	19.9	95	**13.4**	20.1	95	**14.4**	21.7	95	**15.0**	22.6	95	**15.5**	23.3
96	**13.1**	19.7	96	**13.3**	19.9	96	**14.3**	21.5	96	**14.8**	22.3	96	**15.3**	22.9
97	**13.0**	19.5	97	**13.1**	19.7	97	**14.1**	21.2	97	**14.6**	22.0	97	**15.0**	22.6
98	**12.8**	19.3	98	**13.0**	19.5	98	**13.9**	21.0	98	**14.4**	21.7	98	**14.8**	22.3
99	**12.7**	19.1	99	**12.9**	19.3	99	**13.8**	20.7	99	**14.2**	21.4	99	**14.6**	22.0
100	**12.6**	18.9	100	**12.7**	19.1	100	**13.6**	20.5	100	**14.1**	21.1	100	**14.4**	21.7
101	**12.4**	18.7	101	**12.6**	18.9	101	**13.4**	20.2	101	**13.9**	20.8	101	**14.2**	21.3
102	**12.3**	18.5	102	**12.5**	18.7	102	**13.3**	20.0	102	**13.7**	20.6	102	**14.0**	21.0
103	**12.2**	18.3	103	**12.3**	18.5	103	**13.1**	19.7	103	**13.5**	20.3	103	**13.8**	20.7
104	**12.1**	18.1	104	**12.2**	18.3	104	**12.9**	19.5	104	**13.3**	20.0	104	**13.6**	20.4
105	**11.9**	17.9	105	**12.1**	18.1	105	**12.8**	19.2	105	**13.1**	19.7	105	**13.4**	20.1
106	**11.8**	17.7	106	**11.9**	17.9	106	**12.6**	19.0	106	**12.9**	19.4	106	**13.2**	19.8
107	**11.7**	17.5	107	**11.8**	17.7	107	**12.4**	18.7	107	**12.8**	19.2	107	**13.0**	19.5
108	**11.5**	17.3	108	**11.7**	17.5	108	**12.3**	18.5	108	**12.6**	18.9	108	**12.8**	19.2
109	**11.4**	17.2	109	**11.5**	17.3	109	**12.1**	18.2	109	**12.4**	18.6	109	**12.6**	18.9
110	**11.3**	17.0	110	**11.4**	17.1	110	**12.0**	18.0	110	**12.2**	18.3	110	**12.4**	18.6
111	**11.2**	16.8	111	**11.3**	16.9	111	**11.8**	17.7	111	**12.0**	18.1	111	**12.2**	18.3
112	**11.0**	16.6	112	**11.1**	16.7	112	**11.6**	17.5	112	**11.8**	17.8	112	**12.0**	18.0
113	**10.9**	16.4	113	**11.0**	16.5	113	**11.5**	17.3	113	**11.7**	17.5	113	**11.8**	17.7
114	**10.8**	16.2	114	**10.9**	16.3	114	**11.3**	17.0	114	**11.5**	17.3	114	**11.6**	17.4
115	**10.7**	16.0	115	**10.7**	16.2	115	**11.2**	16.8	115	**11.3**	17.0	115	**11.4**	17.1
116	**10.5**	15.8	116	**10.6**	16.0	116	**11.0**	16.5	116	**11.1**	16.7	116	**11.2**	16.8
117	**10.4**	15.6	117	**10.5**	15.8	117	**10.8**	16.3	117	**11.0**	16.5	117	**11.0**	16.5
118	**10.3**	15.5	118	**10.4**	15.6	118	**10.7**	16.1	118	**10.8**	16.2	118	**10.8**	16.2
119	**10.2**	15.3	119	**10.2**	15.4	119	**10.5**	15.8	119	**10.6**	16.0	119	**10.6**	16.0
120	**10.0**	15.1	120	**10.1**	15.2	120	**10.4**	15.6	120	**10.4**	15.7	120	**10.4**	15.7

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–22 (continued)
Available Critical Stress for Compression Members

$F_y = 35$ksi			$F_y = 36$ksi			$F_y = 42$ksi			$F_y = 46$ksi			$F_y = 50$ksi		
$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi		ksi	ksi		ksi	ksi		ksi	ksi		ksi	ksi
	ASD	LRFD		ASD	LRFD		ASD	LRFD		ASD	LRFD		ASD	LRFD
121	9.91	14.9	121	10.0	15.0	121	10.2	15.4	121	10.3	15.4	121	10.3	15.4
122	9.79	14.7	122	9.85	14.8	122	10.1	15.2	122	10.1	15.2	122	10.1	15.2
123	9.67	14.5	123	9.72	14.6	123	9.93	14.9	123	9.94	14.9	123	9.94	14.9
124	9.55	14.3	124	9.59	14.4	124	9.78	14.7	124	9.78	14.7	124	9.78	14.7
125	9.43	14.2	125	9.47	14.2	125	9.62	14.5	125	9.62	14.5	125	9.62	14.5
126	9.31	14.0	126	9.35	14.0	126	9.47	14.2	126	9.47	14.2	126	9.47	14.2
127	9.19	13.8	127	9.22	13.9	127	9.32	14.0	127	9.32	14.0	127	9.32	14.0
128	9.07	13.6	128	9.10	13.7	128	9.17	13.8	128	9.17	13.8	128	9.17	13.8
129	8.95	13.4	129	8.98	13.5	129	9.03	13.6	129	9.03	13.6	129	9.03	13.6
130	8.83	13.3	130	8.86	13.3	130	8.89	13.4	130	8.89	13.4	130	8.89	13.4
131	8.71	13.1	131	8.73	13.1	131	8.76	13.2	131	8.76	13.2	131	8.76	13.2
132	8.60	12.9	132	8.61	12.9	132	8.63	13.0	132	8.63	13.0	132	8.63	13.0
133	8.48	12.7	133	8.49	12.8	133	8.50	12.8	133	8.50	12.8	133	8.50	12.8
134	8.37	12.6	134	8.37	12.6	134	8.37	12.6	134	8.37	12.6	134	8.37	12.6
135	8.25	12.4	135	8.25	12.4	135	8.25	12.4	135	8.25	12.4	135	8.25	12.4
136	8.13	12.2	136	8.13	12.2	136	8.13	12.2	136	8.13	12.2	136	8.13	12.2
137	8.01	12.0	137	8.01	12.0	137	8.01	12.0	137	8.01	12.0	137	8.01	12.0
138	7.89	11.9	138	7.89	11.9	138	7.89	11.9	138	7.89	11.9	138	7.89	11.9
139	7.78	11.7	139	7.78	11.7	139	7.78	11.7	139	7.78	11.7	139	7.78	11.7
140	7.67	11.5	140	7.67	11.5	140	7.67	11.5	140	7.67	11.5	140	7.67	11.5
141	7.56	11.4	141	7.56	11.4	141	7.56	11.4	141	7.56	11.4	141	7.56	11.4
142	7.45	11.2	142	7.45	11.2	142	7.45	11.2	142	7.45	11.2	142	7.45	11.2
143	7.35	11.0	143	7.35	11.0	143	7.35	11.0	143	7.35	11.0	143	7.35	11.0
144	7.25	10.9	144	7.25	10.9	144	7.25	10.9	144	7.25	10.9	144	7.25	10.9
145	7.15	10.7	145	7.15	10.7	145	7.15	10.7	145	7.15	10.7	145	7.15	10.7
146	7.05	10.6	146	7.05	10.6	146	7.05	10.6	146	7.05	10.6	146	7.05	10.6
147	6.96	10.5	147	6.96	10.5	147	6.96	10.5	147	6.96	10.5	147	6.96	10.5
148	6.86	10.3	148	6.86	10.3	148	6.86	10.3	148	6.86	10.3	148	6.86	10.3
149	6.77	10.2	149	6.77	10.2	149	6.77	10.2	149	6.77	10.2	149	6.77	10.2
150	6.68	10.0	150	6.68	10.0	150	6.68	10.0	150	6.68	10.0	150	6.68	10.0
151	6.59	9.91	151	6.59	9.91	151	6.59	9.91	151	6.59	9.91	151	6.59	9.91
152	6.51	9.78	152	6.51	9.78	152	6.51	9.78	152	6.51	9.78	152	6.51	9.78
153	6.42	9.65	153	6.42	9.65	153	6.42	9.65	153	6.42	9.65	153	6.42	9.65
154	6.34	9.53	154	6.34	9.53	154	6.34	9.53	154	6.34	9.53	154	6.34	9.53
155	6.26	9.40	155	6.26	9.40	155	6.26	9.40	155	6.26	9.40	155	6.26	9.40
156	6.18	9.28	156	6.18	9.28	156	6.18	9.28	156	6.18	9.28	156	6.18	9.28
157	6.10	9.17	157	6.10	9.17	157	6.10	9.17	157	6.10	9.17	157	6.10	9.17
158	6.02	9.05	158	6.02	9.05	158	6.02	9.05	158	6.02	9.05	158	6.02	9.05
159	5.95	8.94	159	5.95	8.94	159	5.95	8.94	159	5.95	8.94	159	5.95	8.94
160	5.87	8.82	160	5.87	8.82	160	5.87	8.82	160	5.87	8.82	160	5.87	8.82

ASD	LRFD
$\Omega_c = 1.67$	$\phi_c = 0.90$

Table 4–22 (continued)
Available Critical Stress for Compression Members

F_y = 35ksi			F_y = 36ksi			F_y = 42ksi			F_y = 46ksi			F_y = 50ksi		
$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$	$\frac{Kl}{r}$	F_{cr}/Ω_c	$\phi_c F_{cr}$
	ksi	ksi		ksi	ksi		ksi	ksi		ksi	ksi		ksi	ksi
	ASD	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**		**ASD**	**LRFD**
161	**5.80**	8.72	161	**5.80**	8.72	161	**5.80**	8.72	161	**5.80**	8.72	161	**5.80**	8.72
162	**5.73**	8.61	162	**5.73**	8.61	162	**5.73**	8.61	162	**5.73**	8.61	162	**5.73**	8.61
163	**5.66**	8.50	163	**5.66**	8.50	163	**5.66**	8.50	163	**5.66**	8.50	163	**5.66**	8.50
164	**5.59**	8.40	164	**5.59**	8.40	164	**5.59**	8.40	164	**5.59**	8.40	164	**5.59**	8.40
165	**5.52**	8.30	165	**5.52**	8.30	165	**5.52**	8.30	165	**5.52**	8.30	165	**5.52**	8.30
166	**5.45**	8.20	166	**5.45**	8.20	166	**5.45**	8.20	166	**5.45**	8.20	166	**5.45**	8.20
167	**5.39**	8.10	167	**5.39**	8.10	167	**5.39**	8.10	167	**5.39**	8.10	167	**5.39**	8.10
168	**5.33**	8.00	168	**5.33**	8.00	168	**5.33**	8.00	168	**5.33**	8.00	168	**5.33**	8.00
169	**5.25**	7.89	169	**5.25**	7.89	169	**5.25**	7.89	169	**5.25**	7.89	169	**5.25**	7.89
170	**5.20**	7.82	170	**5.20**	7.82	170	**5.20**	7.82	170	**5.20**	7.82	170	**5.20**	7.82
171	**5.14**	7.73	171	**5.14**	7.73	171	**5.14**	7.73	171	**5.14**	7.73	171	**5.14**	7.73
172	**5.08**	7.64	172	**5.08**	7.64	172	**5.08**	7.64	172	**5.08**	7.64	172	**5.08**	7.64
173	**5.02**	7.55	173	**5.02**	7.55	173	**5.02**	7.55	173	**5.02**	7.55	173	**5.02**	7.55
174	**4.96**	7.46	174	**4.96**	7.46	174	**4.96**	7.46	174	**4.96**	7.46	174	**4.96**	7.46
175	**4.91**	7.38	175	**4.91**	7.38	175	**4.91**	7.38	175	**4.91**	7.38	175	**4.91**	7.38
176	**4.85**	7.29	176	**4.85**	7.29	176	**4.85**	7.29	176	**4.85**	7.29	176	**4.85**	7.29
177	**4.80**	7.21	177	**4.80**	7.21	177	**4.80**	7.21	177	**4.80**	7.21	177	**4.80**	7.21
178	**4.74**	7.13	178	**4.74**	7.13	178	**4.74**	7.13	178	**4.74**	7.13	178	**4.74**	7.13
179	**4.69**	7.05	179	**4.69**	7.05	179	**4.69**	7.05	179	**4.69**	7.05	179	**4.69**	7.05
180	**4.64**	6.97	180	**4.64**	6.97	180	**4.64**	6.97	180	**4.64**	6.97	180	**4.64**	6.97
181	**4.59**	6.90	181	**4.59**	6.90	181	**4.59**	6.90	181	**4.59**	6.90	181	**4.59**	6.90
182	**4.54**	6.82	182	**4.54**	6.82	182	**4.54**	6.82	182	**4.54**	6.82	182	**4.54**	6.82
183	**4.49**	6.75	183	**4.49**	6.75	183	**4.49**	6.75	183	**4.49**	6.75	183	**4.49**	6.75
184	**4.44**	6.67	184	**4.44**	6.67	184	**4.44**	6.67	184	**4.44**	6.67	184	**4.44**	6.67
185	**4.39**	6.60	185	**4.39**	6.60	185	**4.39**	6.60	185	**4.39**	6.60	185	**4.39**	6.60
186	**4.34**	6.53	186	**4.34**	6.53	186	**4.34**	6.53	186	**4.34**	6.53	186	**4.34**	6.53
187	**4.30**	6.46	187	**4.30**	6.46	187	**4.30**	6.46	187	**4.30**	6.46	187	**4.30**	6.46
188	**4.25**	6.39	188	**4.25**	6.39	188	**4.25**	6.39	188	**4.25**	6.39	188	**4.25**	6.39
189	**4.21**	6.32	189	**4.21**	6.32	189	**4.21**	6.32	189	**4.21**	6.32	189	**4.21**	6.32
190	**4.16**	6.26	190	**4.16**	6.26	190	**4.16**	6.26	190	**4.16**	6.26	190	**4.16**	6.26
191	**4.12**	6.19	191	**4.12**	6.19	191	**4.12**	6.19	191	**4.12**	6.19	191	**4.12**	6.19
192	**4.08**	6.13	192	**4.08**	6.13	192	**4.08**	6.13	192	**4.08**	6.13	192	**4.08**	6.13
193	**4.04**	6.06	193	**4.04**	6.06	193	**4.04**	6.06	193	**4.04**	6.06	193	**4.04**	6.06
194	**3.99**	6.00	194	**3.99**	6.00	194	**3.99**	6.00	194	**3.99**	6.00	194	**3.99**	6.00
195	**3.95**	5.94	195	**3.95**	5.94	195	**3.95**	5.94	195	**3.95**	5.94	195	**3.95**	5.94
196	**3.91**	5.88	196	**3.91**	5.88	196	**3.91**	5.88	196	**3.91**	5.88	196	**3.91**	5.88
197	**3.87**	5.82	197	**3.87**	5.82	197	**3.87**	5.82	197	**3.87**	5.82	197	**3.87**	5.82
198	**3.83**	5.76	198	**3.83**	5.76	198	**3.83**	5.76	198	**3.83**	5.76	198	**3.83**	5.76
199	**3.80**	5.70	199	**3.80**	5.70	199	**3.80**	5.70	199	**3.80**	5.70	199	**3.80**	5.70
200	**3.76**	5.65	200	**3.76**	5.65	200	**3.76**	5.65	200	**3.76**	5.65	200	**3.76**	5.65

ASD	**LRFD**
$\Omega_c = 1.67$	$\phi_c = 0.90$

PART 5

DESIGN OF TENSION MEMBERS

SCOPE 5–2

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of members subject to static axial tension. For fatigue applications, see AISC Specification Appendix 3. For the design of members subject to eccentric tension or combined tension and flexure, see Part 6. For tension members that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

GROSS AREA, NET AREA, AND EFFECTIVE NET AREA

In the determination of the available strength of a tension member, the gross area, A_g, is needed for the tensile yielding limit state and the effective net area, A_e, is needed for the tensile rupture limit state.

Gross Area

The gross area, A_g, is determined as specified in AISC Specification Section D3.1.

Effective Net Area

The effective net area, A_e, is determined by multiplying the net area, A_n, by the shear lag coefficient, U, where A_n is determined for tension members per AISC Specification Section D3 and U is determined from AISC Specification Table D3.1. Shear lag parameters are illustrated in AISC Commentary Figure C-D3.1.

TENSILE STRENGTH

The limit-state of tensile yielding will control the available tensile strength over tensile rupture when the following relationship is satisfied:

LRFD	ASD
$0.9F_yA_g \leq 0.75F_uA_e$	$\dfrac{F_yA_g}{1.67} \leq \dfrac{F_uA_e}{2}$

These expressions are both reduced to:

$$\frac{A_e}{A_g} \geq 1.2\frac{F_y}{F_u}$$

Otherwise, the limit-state of tensile rupture will control over tensile yielding.

Yielding Limit State

The available tensile strength due to tensile yielding, which must equal or exceed the required strength, P_u or P_a, is determined for tension members, per AISC Specification Section D2(a), using Equation D2-1.

Rupture Limit State

The available tensile strength due to tensile rupture, which must equal or exceed the required strength, P_u or P_a, is determined for tension members, per AISC Specification Section D2(b) using Equation D2-2.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

Special Requirements for Heavy Shapes and Plates

For tension members with complete-joint-penetration groove welded joints and made from heavy shapes with a flange thickness exceeding 2 in. or built-up sections consisting of plates with a thickness exceeding 2 in., see AISC Specification Sections A3.1c and Section A3.1d.

Slenderness

Tension member slenderness, L/r, should preferably be limited to a maximum of 300 per the User Note in AISC Specification Section D1. The intent of this recommendation is explained in the corresponding Commentary.

STEEL TENSION MEMBER SELECTION TABLES

Available tensile strengths for various types of tension members (see individual descriptions below) are given in Tables 5–1 through 5–8 for the limit states of tensile yielding and tensile rupture. In each case, the tabulated values for available tensile rupture strength are based upon the assumption that $A_e = 0.75A_g$, which is arbitrarily selected as a value that is practical to achieve with typical end connections. Such consideration of the effective net area during the design of the member will simplify the design of its end connections, which can be difficult to configure and costly if tension members are selected based upon available tensile yielding strength only, without considering the reduction in strength due to the connection.

When $A_e > 0.75A_g$, either the tabulated values for available tensile rupture strength can be used conservatively or the available tensile rupture strength can be calculated based upon the actual value of A_e. When $A_e < 0.75A_g$, the tabulated values of the available tensile rupture strength cannot be used, but rather, must be calculated based upon the actual value of A_e.

Table 5–1. W-Shapes

Available strengths in axial tension are given for W-shapes with $F_y = 50$ ksi and $F_u = 65$ ksi (ASTM A992). Note that tensile rupture will control over tensile yielding given for W-shapes with $F_y = 50$ ksi and $F_u = 65$ ksi when $A_e/A_g < 0.923$. Otherwise, tensile yielding will control over tensile rupture.

Table 5–2. Single Angles

Available strengths in axial tension are given for single angles with $F_y = 36$ ksi and $F_u = 58$ ksi (ASTM A36). Note that tensile rupture will control over tensile yielding given for single angles with $F_y = 36$ ksi and $F_u = 58$ ksi when $A_e/A_g < 0.745$. Otherwise, tensile yielding will control over tensile rupture.

Table 5–3. WT-Shapes

Table 5–3 is similar to Table 5–1, except it covers WT-shapes with $F_y = 50$ ksi and $F_u = 65$ ksi (ASTM A992).

Table 5–4. Rectangular HSS

Available strengths in axial tension are given for rectangular HSS with $F_y = 46$ ksi and $F_u = 58$ ksi (ASTM A500 Grade B). Note that tensile rupture will control over tensile yielding given for rectangular HSS with $F_y = 46$ ksi and $F_u = 58$ ksi when $A_e/A_g < 0.952$. Otherwise, tensile yielding will control over tensile rupture.

Table 5–5. Square HSS

Table 5–3 is similar to Table 5–1, except it covers square HSS with $F_y = 46$ ksi and $F_u = 58$ ksi (ASTM A500 Grade B).

Table 5–6. Round HSS

Available strengths in axial tension are given for ASTM A500 round HSS with $F_y = 42$ ksi and $F_u = 58$ ksi (ASTM A500 Grade B). Note that tensile rupture will control over tensile yielding given for round HSS with $F_y = 42$ ksi and $F_u = 58$ ksi when $A_e/A_g < 0.869$. Otherwise, tensile yielding will control over tensile rupture.

Table 5–7. Steel Pipe

Available strengths in axial tension are given for steel pipe with $F_y = 35$ ksi and $F_u = 60$ ksi (ASTM A53 grade B). Note that tensile rupture will control over tensile yielding given for steel pipe with $F_y = 35$ ksi and $F_u = 60$ ksi when $A_e/A_g < 0.700$. Otherwise, tensile yielding will control over tensile rupture.

Table 5–8. Double Angles

Available strengths in axial tension are given for double angles with $F_y = 36$ ksi and $F_u = 58$ ksi (ASTM A36). Note that tensile rupture will control over tensile yielding given for double angles with $F_y = 36$ ksi and $F_u = 58$ ksi when $A_e/A_g < 0.745$. Otherwise, tensile yielding will control over tensile rupture.

F_y = 50 ksi
F_u = 65 ksi

Table 5-1
Available Strength in Axial Tension
W Shapes

W44–W40

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	**ASD**	**LRFD**	**ASD**	**LRFD**
W44×335	98.5	73.9	**2950**	4430	**2400**	3600
×290	85.4	64.1	**2560**	3840	**2080**	3120
×262	76.9	57.7	**2300**	3460	**1880**	2810
×230	67.7	50.8	**2030**	3050	**1650**	2480
W40×593^h	174	131	**5210**	7830	**4260**	6390
×503^h	148	111	**4430**	6660	**3610**	5410
×431^h	127	95.3	**3800**	5720	**3100**	4650
×397^h	117	87.8	**3500**	5270	**2850**	4280
×372^h	109	81.8	**3260**	4910	**2660**	3990
×362^h	107	80.3	**3200**	4820	**2610**	3910
×324	95.3	71.5	**2850**	4290	**2320**	3490
×297	87.4	65.6	**2620**	3930	**2130**	3200
×277	81.4	61.0	**2440**	3660	**1980**	2970
×249	73.3	55.0	**2190**	3300	**1790**	2680
×215	63.4	47.6	**1900**	2850	**1550**	2320
×199	58.5	43.9	**1750**	2630	**1430**	2140
W40×392^h	115	86.3	**3440**	5180	**2800**	4210
×331^h	97.5	73.1	**2920**	4390	**2380**	3560
×327^h	96.0	72.0	**2870**	4320	**2340**	3510
×294	86.3	64.7	**2580**	3880	**2100**	3150
×278	82.0	61.5	**2460**	3690	**2000**	3000
×264	77.6	58.2	**2320**	3490	**1890**	2840
×235	69.0	51.8	**2070**	3110	**1680**	2530
×211	62.0	46.5	**1860**	2790	**1510**	2270
×183	53.3	40.0	**1600**	2400	**1300**	1950
×167	49.2	36.9	**1470**	2210	**1200**	1800
×149	43.8	32.8	**1310**	1970	**1070**	1600

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

Table 5-1 (continued)

Available Strength in Axial Tension

W Shapes

F_y = 50 ksi
F_u = 65 ksi

W36–W33

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
W36×800[h]	236	177	**7070**	10600	**5750**	8630
×652[h]	192	144	**5750**	8640	**4680**	7020
×529[h]	156	117	**4670**	7020	**3800**	5700
×487[h]	143	107.0	**4280**	6440	**3480**	5220
×441[h]	130	97.5	**3890**	5850	**3170**	4750
×395[h]	116	87.0	**3470**	5220	**2830**	4240
×361[h]	106.0	79.5	**3170**	4770	**2580**	3880
×330	97.0	72.8	**2900**	4370	**2370**	3550
×302	88.8	66.6	**2660**	4000	**2160**	3250
×282	82.9	62.2	**2480**	3730	**2020**	3030
×262	77.0	57.8	**2310**	3470	**1880**	2820
×247	72.5	54.4	**2170**	3260	**1770**	2650
×231	68.1	51.1	**2040**	3060	**1660**	2490
W36×256	75.4	56.6	**2260**	3390	**1840**	2760
×232	68.1	51.1	**2040**	3060	**1660**	2490
×210	61.8	46.3	**1850**	2780	**1500**	2260
×194	57.0	42.7	**1710**	2560	**1390**	2080
×182	53.6	40.2	**1600**	2410	**1310**	1960
×170	50.1	37.6	**1500**	2250	**1220**	1830
×160	47.0	35.3	**1410**	2120	**1150**	1720
×150	44.2	33.2	**1320**	1990	**1080**	1620
×135	39.7	29.8	**1190**	1790	**968**	1450
W33×387[h]	114	85.5	**3410**	5130	**2780**	4170
×354[h]	104	78.0	**3110**	4680	**2540**	3800
×318	93.6	70.2	**2800**	4210	**2280**	3420
×291	85.7	64.3	**2570**	3860	**2090**	3130
×263	77.5	58.1	**2320**	3490	**1890**	2830
×241	71.0	53.3	**2130**	3200	**1730**	2600
×221	65.2	48.9	**1950**	2930	**1590**	2380
×201	59.2	44.4	**1770**	2660	**1440**	2160

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-1 (continued)
Available Strength in Axial Tension
W Shapes

W33–W27

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	**ASD**	LRFD	**ASD**	LRFD
W33×169	49.5	37.1	**1480**	2230	**1210**	1810
×152	44.8	33.6	**1340**	2020	**1090**	1640
×141	41.6	31.2	**1250**	1870	**1010**	1520
×130	38.3	28.7	**1150**	1720	**933**	1400
×118	34.7	26.0	**1040**	1560	**845**	1270
W30×391^h	115	86.3	**3440**	5180	**2800**	4210
×357^h	105	78.8	**3140**	4730	**2560**	3840
×326^h	95.8	71.8	**2870**	4310	**2330**	3500
×292	85.9	64.4	**2570**	3870	**2090**	3140
×261	76.9	57.7	**2300**	3460	**1880**	2810
×235	69.2	51.9	**2070**	3110	**1690**	2530
×211	62.2	46.7	**1860**	2800	**1520**	2280
×191	56.3	42.2	**1690**	2530	**1370**	2060
×173	51.0	38.3	**1530**	2300	**1240**	1870
W30×148	43.5	32.6	**1300**	1960	**1060**	1590
×132	38.9	29.2	**1160**	1750	**949**	1420
×124	36.5	27.4	**1090**	1640	**891**	1340
×116	34.2	25.7	**1020**	1540	**835**	1250
×108	31.7	23.8	**949**	1430	**773**	1160
×99	29.1	21.8	**871**	1310	**709**	1060
×90	26.4	19.8	**790**	1190	**644**	965
W27×539^h	159	119	**4760**	7160	**3870**	5800
×368^h	108	81.0	**3230**	4860	**2630**	3950
×336^h	98.9	74.2	**2960**	4450	**2410**	3620
×307^h	90.4	67.8	**2710**	4070	**2200**	3310
×281	82.9	62.2	**2480**	3730	**2020**	3030
×258	76.0	57.0	**2280**	3420	**1850**	2780
×235	69.4	52.1	**2080**	3120	**1690**	2540
×217	64.0	48.0	**1920**	2880	**1560**	2340
×194	57.2	42.9	**1710**	2570	**1390**	2090
×178	52.5	39.4	**1570**	2360	**1280**	1920
×161	47.6	35.7	**1430**	2140	**1160**	1740
×146	43.1	32.3	**1290**	1940	**1050**	1570

Limit State	**ASD**	**LRFD**
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

W27–W21

Table 5-1 (continued)
Available Strength in Axial Tension
W Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	ϕ_tP_n	P_n/Ω_t	ϕ_tP_n
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
W27×129	37.8	28.4	1130	1700	923	1380
×114	33.5	25.1	1000	1510	816	1220
×102	30.0	22.5	898	1350	731	1100
×94	27.7	20.8	829	1250	676	1010
×84	24.8	18.6	743	1120	605	907
W24×370^h	109	81.8	3260	4910	2660	3990
×335^h	98.4	73.8	2950	4430	2400	3600
×306^h	89.8	67.4	2690	4040	2190	3290
×279^h	82.0	61.5	2460	3690	2000	3000
×250	73.5	55.1	2200	3310	1790	2690
×229	67.2	50.4	2010	3020	1640	2460
×207	60.7	45.5	1820	2730	1480	2220
×192	56.3	42.2	1690	2530	1370	2060
×176	51.7	38.8	1550	2330	1260	1890
×162	47.7	35.8	1430	2150	1160	1750
×146	43.0	32.3	1290	1940	1050	1570
×131	38.5	28.9	1150	1730	939	1410
×117	34.4	25.8	1030	1550	839	1260
×104	30.6	22.9	916	1380	744	1120
W24×103	30.3	22.7	907	1360	738	1110
×94	27.7	20.8	829	1250	676	1010
×84	24.7	18.5	740	1110	601	902
×76	22.4	16.8	671	1010	546	819
×68	20.1	15.1	602	905	491	736
×62	18.2	13.6	545	819	442	663
×55	16.2	12.1	485	729	393	590
W21×201	59.2	44.4	1770	2660	1440	2160
×182	53.6	40.2	1600	2410	1310	1960
×166	48.8	36.6	1460	2200	1190	1780
×147	43.2	32.4	1290	1940	1050	1580
×132	38.8	29.1	1160	1750	946	1420
×122	35.9	26.9	1070	1620	874	1310
×111	32.7	24.5	979	1470	796	1190
×101	29.8	22.3	892	1340	725	1090

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5–1 (continued)
Available Strength in Axial Tension
W Shapes

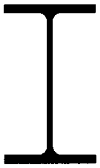
W21–W18

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W21×93	27.3	20.5	**817**	1230	**666**	999
×83	24.3	18.2	**728**	1090	**592**	887
×73	21.5	16.1	**644**	968	**523**	785
×68	20.0	15.0	**599**	900	**488**	731
×62	18.3	13.7	**548**	824	**445**	668
×55	16.2	12.1	**485**	729	**393**	590
×48	14.1	10.6	**422**	634	**345**	517
W21×57	16.7	12.5	**500**	752	**406**	609
×50	14.7	11.0	**440**	662	**358**	536
×44	13.0	9.75	**389**	585	**317**	475
W18×311	91.6	68.7	**2740**	4120	**2230**	3350
×283[h]	83.3	62.5	**2490**	3750	**2030**	3050
×258[h]	75.9	56.9	**2270**	3420	**1850**	2770
×234[h]	68.8	51.6	**2060**	3100	**1680**	2520
×211[h]	62.1	46.6	**1860**	2790	**1510**	2270
×192	56.4	42.3	**1690**	2540	**1370**	2060
×175	51.3	38.5	**1540**	2310	**1250**	1880
×158	46.3	34.7	**1390**	2080	**1130**	1690
×143	42.1	31.6	**1260**	1890	**1030**	1540
×130	38.2	28.7	**1140**	1720	**933**	1400
×119	35.1	26.3	**1050**	1580	**855**	1280
×106	31.1	23.3	**931**	1400	**757**	1140
×97	28.5	21.4	**853**	1280	**696**	1040
×86	25.3	19.0	**757**	1140	**618**	926
×76	22.3	16.7	**668**	1000	**543**	814
W18×71	20.8	15.6	**623**	936	**507**	761
×65	19.1	14.3	**572**	860	**465**	697
×60	17.6	13.2	**527**	792	**429**	644
×55	16.2	12.1	**485**	729	**393**	590
×50	14.7	11.0	**440**	662	**358**	536
W18×46	13.5	10.1	**404**	608	**328**	492
×40	11.8	8.85	**353**	531	**288**	431
×35	10.3	7.73	**308**	463	**251**	377

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.



W16–W14

Table 5–1 (continued)
Available Strength in Axial Tension
W Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
W16×100	29.5	22.1	**883**	1330	**718**	1080
×89	26.2	19.7	**784**	1180	**640**	960
×77	22.6	17.0	**677**	1020	**553**	829
×67	19.7	14.8	**590**	887	**481**	722
W16×57	16.8	12.6	**503**	756	**410**	614
×50	14.7	11.0	**440**	662	**358**	536
×45	13.3	9.98	**398**	599	**324**	487
×40	11.8	8.85	**353**	531	**288**	431
×36	10.6	7.95	**317**	477	**258**	388
W16×31	9.13	6.85	**273**	411	**223**	334
×26	7.68	5.76	**230**	346	**187**	281
W14×730	215	161	**6440**	9680	**5230**	7850
×665^{h}	196	147	**5870**	8820	**4780**	7170
×605^{h}	178	134	**5330**	8010	**4360**	6530
×550^{h}	162	122	**4850**	7290	**3970**	5950
×500^{h}	147	110	**4400**	6620	**3580**	5360
×455^{h}	134	101	**4010**	6030	**3280**	4920
×426^{h}	125	93.8	**3740**	5630	**3050**	4570
×398^{h}	117	87.8	**3500**	5270	**2850**	4280
×370^{h}	109	81.8	**3260**	4910	**2660**	3990
×34^{h}	101	75.8	**3020**	4550	**2460**	3700
×311^{h}	91.4	68.6	**2740**	4110	**2230**	3340
×283^{h}	83.3	62.5	**2490**	3750	**2030**	3050
×257^{h}	75.6	56.7	**2260**	3400	**1840**	2760
×233	68.5	51.4	**2050**	3080	**1670**	2510
×211	62.0	46.5	**1860**	2790	**1510**	2270
×193	56.8	42.6	**1700**	2560	**1380**	2080
×176	51.8	38.9	**1550**	2330	**1260**	1900
×159	46.7	35.0	**1400**	2100	**1140**	1710
×145	42.7	32.0	**1280**	1920	**1040**	1560

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5–1 (continued)
Available Strength in Axial Tension
W Shapes

W14–W12

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
W14×132	38.8	29.1	**1160**	1750	**946**	1420
×120	35.3	26.5	**1060**	1590	**861**	1290
×109	32.0	24.0	**958**	1440	**780**	1170
×99	29.1	21.8	**871**	1310	**709**	1060
×90	26.5	19.9	**793**	1190	**647**	970
W14×82	24.0	18.0	**719**	1080	**585**	878
×74	21.8	16.4	**653**	981	**533**	800
×68	20.0	15.0	**599**	900	**488**	731
×61	17.9	13.4	**536**	805	**436**	653
W14×53	15.6	11.7	**467**	702	**380**	570
×48	14.1	10.6	**422**	634	**345**	517
×43	12.6	9.45	**377**	567	**307**	461
W14×38	11.2	8.40	**335**	504	**273**	410
×34	10.0	7.50	**299**	450	**244**	366
×30	8.85	6.64	**265**	398	**216**	324
W14×26	7.69	5.77	**230**	346	**188**	281
×22	6.49	4.87	**194**	292	**158**	237
W12×336	98.8	74.1	**2960**	4450	**2410**	3610
×305[h]	89.6	67.2	**2680**	4030	**2180**	3280
×279[h]	81.9	61.4	**2450**	3690	**2000**	2990
×252[h]	74.0	55.5	**2220**	3330	**1800**	2710
×230[h]	67.7	50.8	**2030**	3050	**1650**	2480
×210[h]	61.8	46.3	**1850**	2780	**1500**	2260
×190	55.8	41.9	**1670**	2510	**1360**	2040
×170	50.0	37.5	**1500**	2250	**1220**	1830
×152	44.7	33.5	**1340**	2010	**1090**	1630
×136	39.9	29.9	**1190**	1800	**972**	1460
×120	35.3	26.5	**1060**	1590	**861**	1290
×106	31.2	23.4	**934**	1400	**761**	1140
×96	28.2	21.2	**844**	1270	**689**	1030
×87	25.6	19.2	**766**	1150	**624**	936
×79	23.2	17.4	**695**	1040	**566**	848
×72	21.1	15.8	**632**	949	**514**	770
×65	19.1	14.3	**572**	860	**465**	697

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

W12–W10

Table 5-1 (continued)
Available Strength in Axial Tension
W Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
W12×58	17.0	12.8	**509**	765	**416**	624
×53	15.6	11.7	**467**	702	**380**	570
W12×50	14.6	10.9	**437**	657	**354**	531
×45	13.1	9.83	**392**	590	**319**	479
×40	11.7	8.78	**350**	527	**285**	428
W12×35	10.3	7.73	**308**	463	**251**	377
×30	8.79	6.59	**263**	396	**214**	321
×26	7.65	5.74	**229**	344	**187**	280
W12×22	6.48	4.86	**194**	292	**158**	237
×19	5.57	4.18	**167**	251	**136**	204
×16	4.71	3.53	**141**	212	**115**	172
×14	4.16	3.12	**125**	187	**101**	152
W10×112	32.9	24.7	**985**	1480	**803**	1200
×100	29.4	22.0	**880**	1320	**715**	1070
×88	25.9	19.4	**775**	1170	**631**	946
×77	22.6	17.0	**677**	1020	**553**	829
×68	20.0	15.0	**599**	900	**488**	731
×60	17.6	13.2	**527**	792	**429**	644
×54	15.8	11.9	**473**	711	**387**	580
×49	14.4	10.8	**431**	648	**351**	527
W10×45	13.3	9.98	**398**	599	**324**	487
×39	11.5	8.63	**344**	518	**280**	421
×33	9.71	7.28	**291**	437	**237**	355
W10×30	8.84	6.63	**265**	398	**215**	323
×26	7.61	5.71	**228**	342	**186**	278
×22	6.49	4.87	**194**	292	**158**	237
W10×19	5.62	4.22	**168**	253	**137**	206
×17	4.99	3.74	**149**	225	**122**	182
×15	4.41	3.31	**132**	198	**108**	161
×12	3.54	2.66	**106**	159	**86.5**	130

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-1 (continued)
Available Strength in Axial Tension
W Shapes

W8

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
W8×67	19.7	14.8	590	887	481	722
×58	17.1	12.8	512	770	416	624
×48	14.1	10.6	422	634	345	517
×40	11.7	8.78	350	527	285	428
×35	10.3	7.73	308	463	251	377
×31	9.12	6.84	273	410	222	333
W8×28	8.24	6.18	247	371	201	301
×24	7.08	5.31	212	319	173	259
W8×21	6.16	4.62	184	277	150	225
×18	5.26	3.94	157	237	128	192
W8×15	4.44	3.33	133	200	108	162
×13	3.84	2.88	115	173	93.6	140
×10	2.96	2.22	88.6	133	72.2	108

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

Table 5-2
Available Strength in Axial Tension
Angles

L8–L6

F_y = 36 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
L8×8×1$^1/_8$	16.7	12.5	**360**	541	**363**	544
×1	15.0	11.3	**323**	486	**328**	492
×$^7/_8$	13.2	9.90	**285**	428	**287**	431
×$^3/_4$	11.4	8.55	**246**	369	**248**	372
×$^5/_8$	9.61	7.21	**207**	311	**209**	314
×$^9/_{16}$	8.68	6.51	**187**	281	**189**	283
×$^1/_2$	7.75	5.81	**167**	251	**168**	253
L8×6×1	13.0	9.75	**280**	421	**283**	424
×$^7/_8$	11.5	8.63	**248**	373	**250**	375
×$^3/_4$	9.94	7.46	**214**	322	**216**	325
×$^5/_8$	8.36	6.27	**180**	271	**182**	273
×$^9/_{16}$	7.56	5.67	**163**	245	**164**	247
×$^1/_2$	6.75	5.06	**146**	219	**147**	220
×$^7/_{16}$	5.93	4.45	**128**	192	**129**	194
L8×4×1	11.0	8.25	**237**	356	**239**	359
×$^7/_8$	9.73	7.30	**210**	315	**212**	318
×$^3/_4$	8.44	6.33	**182**	273	**184**	275
×$^5/_8$	7.11	5.33	**153**	230	**155**	232
×$^9/_{16}$	6.43	4.82	**139**	208	**140**	210
×$^1/_2$	5.75	4.31	**124**	186	**125**	187
×$^7/_{16}$	5.06	3.80	**109**	164	**110**	165
L7×4×$^3/_4$	7.69	5.77	**166**	249	**167**	251
×$^5/_8$	6.48	4.86	**140**	210	**141**	211
×$^1/_2$	5.25	3.94	**113**	170	**114**	171
×$^7/_{16}$	4.62	3.47	**99.6**	150	**101**	151
×$^3/_8$	3.98	2.99	**85.8**	129	**86.7**	130
L6×6×1	11.0	8.25	**237**	356	**239**	359
×$^7/_8$	9.75	7.31	**210**	316	**212**	318
×$^3/_4$	8.46	6.34	**182**	274	**184**	276
×$^5/_8$	7.13	5.35	**154**	231	**155**	233
×$^9/_{16}$	6.45	4.84	**139**	209	**140**	211
×$^1/_2$	5.77	4.33	**124**	187	**126**	188
×$^7/_{16}$	5.08	3.81	**110**	165	**110**	166
×$^3/_8$	4.38	3.29	**94.4**	142	**95.4**	143
×$^5/_{16}$	3.67	2.75	**79.1**	119	**79.8**	120

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 36 ksi
F_u = 58 ksi

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

L6–L5

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
L6×4×7/8	7.98	5.99	172	259	174	261
×3/4	6.94	5.21	150	225	151	227
×5/8	5.86	4.39	126	190	127	191
×9/16	5.31	3.98	114	172	115	173
×1/2	4.75	3.56	102	154	103	155
×7/16	4.18	3.14	90.1	135	91.1	137
×3/8	3.61	2.71	77.8	117	78.6	118
×5/16	3.03	2.27	65.3	98.2	65.8	98.7
L6×3½×1/2	4.50	3.38	97.0	146	98.0	147
×3/8	3.42	2.57	73.7	111	74.5	112
×5/16	2.87	2.15	61.9	93.0	62.4	93.5
L5×5×7/8	7.98	5.99	172	259	174	261
×3/4	6.94	5.21	150	225	151	227
×5/8	5.86	4.39	126	190	127	191
×1/2	4.75	3.56	102	154	103	155
×7/16	4.18	3.14	90.1	135	91.1	137
×3/8	3.61	2.71	77.8	117	78.6	118
×5/16	3.03	2.27	65.3	98.2	65.8	98.7
L5×3½×3/4	5.81	4.36	125	188	126	190
×5/8	4.92	3.69	106	159	107	161
×1/2	4.00	3.00	86.2	130	87.0	131
×3/8	3.05	2.29	65.7	98.8	66.4	99.6
×5/16	2.56	1.92	55.2	82.9	55.7	83.5
×1/4	2.06	1.55	44.4	66.7	45.0	67.4
L5×3×1/2	3.75	2.81	80.8	122	81.5	122
×7/16	3.31	2.48	71.4	107	71.9	108
×3/8	2.86	2.15	61.7	92.7	62.4	93.5
×5/16	2.40	1.80	51.7	77.8	52.2	78.3
×1/4	1.94	1.46	41.8	62.9	42.3	63.5

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

L4–L3 1/2

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

F_y = 36 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
L4×4×3/4	5.44	4.08	**117**	176	**118**	177
×5/8	4.61	3.46	**99.4**	149	**100**	151
×1/2	3.75	2.81	**80.8**	122	**81.5**	122
×7/16	3.31	2.48	**71.4**	107	**71.9**	108
×3/8	2.86	2.15	**61.7**	92.7	**62.4**	93.5
×5/16	2.40	1.80	**51.7**	77.8	**52.2**	78.3
×1/4	1.94	1.46	**41.8**	62.9	**42.3**	63.5
L4×3 1/2×1/2	3.50	2.63	**75.4**	113	**76.3**	114
×3/8	2.67	2.00	**57.6**	86.5	**58.0**	87.0
×5/16	2.25	1.69	**48.5**	72.9	**49.0**	73.5
×1/4	1.81	1.36	**39.0**	58.6	**39.4**	59.2
L4×3×5/8	3.89	2.92	**83.9**	126	**84.7**	127
×1/2	3.25	2.44	**70.1**	105	**70.8**	106
×3/8	2.48	1.86	**53.5**	80.4	**53.9**	80.9
×5/16	2.09	1.57	**45.1**	67.7	**45.5**	68.3
×1/4	1.69	1.27	**36.4**	54.8	**36.8**	55.2
L3 1/2×3 1/2×1/2	3.25	2.44	**70.1**	105	**70.8**	106
×7/16	2.87	2.15	**61.9**	93.0	**62.4**	93.5
×3/8	2.48	1.86	**53.5**	80.4	**53.9**	80.9
×5/16	2.09	1.57	**45.1**	67.7	**45.5**	68.3
×1/4	1.69	1.27	**36.4**	54.8	**36.8**	55.2
L3 1/2×3×1/2	3.00	2.25	**64.7**	97.2	**65.3**	97.9
×7/16	2.65	1.99	**57.1**	85.9	**57.7**	86.6
×3/8	2.30	1.73	**49.6**	74.5	**50.2**	75.3
×5/16	1.93	1.45	**41.6**	62.5	**42.1**	63.1
×1/4	1.56	1.17	**33.6**	50.5	**33.9**	50.9
×1/2	2.75	2.06	**59.3**	89.1	**59.7**	89.6
×3/8	2.11	1.58	**45.5**	68.4	**45.8**	68.7
×5/16	1.78	1.34	**38.4**	57.7	**38.9**	58.3
×1/4	1.44	1.08	**31.0**	46.7	**31.3**	47.0

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 36 ksi
F_u = 58 ksi

Table 5-2 (continued)
Available Strength in Axial Tension
Angles

L3–L2

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
L3×3×1/2	2.75	2.06	59.3	89.1	59.7	89.6
×7/16	2.43	1.82	52.4	78.7	52.8	79.2
×3/8	2.11	1.58	45.5	68.4	45.8	68.7
×5/16	1.78	1.34	38.4	57.7	38.9	58.3
×1/4	1.44	1.08	31.0	46.7	31.3	47.0
×3/16	1.09	0.818	23.5	35.3	23.7	35.6
L3×2 1/2×1/2	2.50	1.88	53.9	81.0	54.5	81.8
×7/16	2.21	1.66	47.6	71.6	48.1	72.2
×3/8	1.92	1.44	41.4	62.2	41.8	62.6
×5/16	1.67	1.25	36.0	54.1	36.3	54.4
×1/4	1.31	0.983	28.2	42.4	28.5	42.8
×3/16	1.00	0.747	21.5	32.3	21.7	32.5
L3×2×1/2	2.25	1.69	48.5	72.9	49.0	73.5
×3/8	1.73	1.30	37.3	56.1	37.7	56.6
×5/16	1.46	1.10	31.5	47.3	31.9	47.9
×1/4	1.19	0.892	25.7	38.6	25.9	38.8
×3/16	0.902	0.676	19.4	29.2	19.6	29.4
L2 1/2×2 1/2×1/2	2.25	1.69	48.5	72.9	49.0	73.5
×3/8	1.73	1.30	37.3	56.1	37.7	56.6
×5/16	1.46	1.10	31.5	47.3	31.9	47.9
×1/4	1.19	0.892	25.7	38.6	25.9	38.8
×3/16	0.900	0.675	19.4	29.2	19.6	29.4
L2 1/2×2×3/8	1.55	1.16	33.4	50.2	33.6	50.5
×5/16	1.31	0.983	28.2	42.4	28.5	42.8
×1/4	1.06	0.795	22.9	34.3	23.1	34.6
×3/16	0.809	0.607	17.4	26.2	17.6	26.4
L2 1/2×1 1/2×1/4	0.938	0.704	20.2	30.4	20.4	30.6
×3/16	0.715	0.536	15.4	23.2	15.5	23.3
L2×2×3/8	1.36	1.02	29.3	44.1	29.6	44.4
×5/16	1.15	0.863	24.8	37.3	25.0	37.5
×1/4	0.938	0.704	20.2	30.4	20.4	30.6
×3/16	0.715	0.536	15.4	23.2	15.5	23.3
×1/8	0.484	0.363	10.4	15.7	10.5	15.8

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

WT22–WT20

Table 5-3
Available Strength in Axial Tension
WT Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	**ASD**	LRFD	**ASD**	LRFD
WT22×167.5	49.2	36.9	**1470**	2210	**1200**	1800
×145	42.7	32.0	**1280**	1920	**1040**	1560
×131	38.4	28.8	**1150**	1730	**936**	1400
×115	33.8	25.4	**1010**	1520	**826**	1240
WT20×296.5[h]	87.2	65.4	**2610**	3920	**2130**	3190
×251.5[h]	73.9	55.4	**2210**	3330	**1800**	2700
×215.5[h]	63.4	47.6	**1900**	2850	**1550**	2320
×198.5[h]	58.4	43.8	**1750**	2630	**1420**	2140
×186[h]	54.6	41.0	**1630**	2460	**1330**	2000
×181[h]	53.3	40.0	**1600**	2400	**1300**	1950
×162	47.7	35.8	**1430**	2150	**1160**	1750
×148.5	43.7	32.8	**1310**	1970	**1070**	1600
×138.5	40.7	30.5	**1220**	1830	**991**	1490
×124.5	36.7	27.5	**1100**	1650	**894**	1340
×107.5	31.7	23.8	**949**	1430	**773**	1160
×99.5	29.2	21.9	**874**	1310	**712**	1070
WT20×196[h]	57.6	43.2	**1720**	2590	**1400**	2110
×165.5[h]	48.7	36.5	**1460**	2190	**1190**	1780
×163.5[h]	48.0	36.0	**1440**	2160	**1170**	1760
×147	43.1	32.3	**1290**	1940	**1050**	1570
×139	41.0	30.8	**1230**	1850	**1000**	1500
×132	38.8	29.1	**1160**	1750	**946**	1420
×117.5	34.5	25.9	**1030**	1550	**842**	1260
×105.5	31.0	23.3	**928**	1400	**757**	1140
×91.5	26.7	20.0	**799**	1200	**650**	975
×83.5	24.6	18.5	**737**	1110	**601**	902
×74.5	21.9	16.4	**656**	985	**533**	800

Limit State	ASD	LRFD
Yielding	Ω_t = **1.67**	ϕ_t = **0.90**
Rupture	Ω_t = **2.00**	ϕ_t = **0.75**

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-3 (continued)
Available Strength in Axial Tension
WT Shapes

WT18–WT16.5

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
WT18×400^h	118	88.5	3530	5310	2880	4310
×326^h	96.1	72.1	2880	4320	2340	3510
×264.5^h	77.8	58.3	2330	3500	1890	2840
×243.5^h	71.7	53.8	2150	3230	1750	2620
×220.5^h	64.9	48.7	1940	2920	1580	2370
×197.5^h	58.2	43.7	1740	2620	1420	2130
×180.5^h	53.0	39.8	1590	2390	1290	1940
×165	48.5	36.4	1450	2180	1180	1770
×151	44.4	33.3	1330	2000	1080	1620
×141	41.5	31.1	1240	1870	1010	1520
×131	38.5	28.9	1150	1730	939	1410
×123.5	36.3	27.2	1090	1630	884	1330
×115.5	34.0	25.5	1020	1530	829	1240
WT18×128	37.7	28.3	1130	1700	920	1380
×116	34.1	25.6	1020	1530	832	1250
×105	30.9	23.2	925	1390	754	1130
×97	28.5	21.4	853	1280	696	1040
×91	26.8	20.1	802	1210	653	980
×85	25.0	18.8	749	1130	611	917
×80	23.5	17.6	704	1060	572	858
×75	22.1	16.6	662	995	540	809
×67.5	19.9	14.9	596	896	484	726
WT16.5×193.5^h	57.0	42.7	1710	2560	1390	2080
×177^h	52.1	39.1	1560	2340	1270	1910
×159	46.8	35.1	1400	2110	1140	1710
×145.5	42.8	32.1	1280	1930	1040	1560
×131.5	38.7	29.0	1160	1740	942	1410
×120.5	35.5	26.6	1060	1600	865	1300
×110.5	32.6	24.5	976	1470	796	1190
×100.5	29.6	22.2	886	1330	722	1080

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

WT16.5–WT13.5

Table 5–3 (continued)

Available Strength in Axial Tension

WT Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
WT16.5×84.5	24.8	18.6	**743**	1120	**605**	907
×76	22.4	16.8	**671**	1010	**546**	819
×70.5	20.8	15.6	**623**	936	**507**	761
×65	19.2	14.4	**575**	864	**468**	702
×59	17.3	13.0	**518**	778	**423**	634
WT15×195.5[h]	57.6	43.2	**1720**	2590	**1400**	2110
×178.5[h]	52.5	39.4	**1570**	2360	**1280**	1920
×163[h]	47.9	35.9	**1430**	2160	**1170**	1750
×146	42.9	32.2	**1280**	1930	**1050**	1570
×130.5	38.4	28.8	**1150**	1730	**936**	1400
×117.5	34.6	25.9	**1040**	1560	**842**	1260
×105.5	31.1	23.3	**931**	1400	**757**	1140
×95.5	28.1	21.1	**841**	1260	**686**	1030
×86.5	25.5	19.1	**763**	1150	**621**	931
WT15×74	21.7	16.3	**650**	977	**530**	795
×66	19.4	14.6	**581**	873	**475**	712
×62	18.2	13.6	**545**	819	**442**	663
×58	17.1	12.8	**512**	770	**416**	624
×54	15.9	11.9	**476**	716	**387**	580
×49.5	14.5	10.9	**434**	652	**354**	531
×45	13.2	9.90	**395**	594	**322**	483
WT13.5×269.5[h]	79.3	59.5	**2370**	3570	**1930**	2900
×184[h]	54.2	40.7	**1620**	2440	**1320**	1980
×168[h]	49.5	37.1	**1480**	2230	**1210**	1810
×153.5[h]	45.2	33.9	**1350**	2030	**1100**	1650
×140.5	41.4	31.1	**1240**	1860	**1010**	1520
×129	38.0	28.5	**1140**	1710	**926**	1390
×117.5	34.7	26.0	**1040**	1560	**845**	1270
×108.5	32.0	24.0	**958**	1440	**780**	1170
×97	28.6	21.5	**856**	1290	**699**	1050
×89	26.2	19.7	**784**	1180	**640**	960
×80.5	23.8	17.9	**713**	1070	**582**	873
×73	21.6	16.2	**647**	972	**527**	790

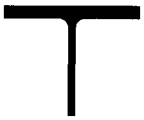
Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5–3 (continued)
Available Strength in Axial Tension
WT Shapes



WT13.5–WT10.5

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding, kips		Rupture, kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
WT13.5×64.5	18.9	14.2	**566**	850	**462**	692
×57	16.8	12.6	**503**	756	**410**	614
×51	15.0	11.3	**449**	675	**367**	551
×47	13.8	10.4	**413**	621	**338**	507
×42	12.4	9.30	**371**	558	**302**	453
WT12×185[h]	54.4	40.8	**1630**	2450	**1330**	1990
×167.5[h]	49.2	36.9	**1470**	2210	**1200**	1800
×153[h]	44.9	33.7	**1340**	2020	**1100**	1640
×139.5[h]	41.0	30.8	**1230**	1850	**1000**	1500
×125	36.8	27.6	**1100**	1660	**897**	1350
×114.5	33.6	25.2	**1010**	1510	**819**	1230
×103.5	30.4	22.8	**910**	1370	**741**	1110
×96	28.1	21.1	**841**	1260	**686**	1030
×88	25.8	19.4	**772**	1160	**631**	946
×81	23.9	17.9	**716**	1080	**582**	873
×73	21.5	16.1	**644**	968	**523**	785
×65.5	19.3	14.5	**578**	869	**471**	707
×58.5	17.2	12.9	**515**	774	**419**	629
×52	15.3	11.5	**458**	689	**374**	561
WT12×51.5	15.1	11.3	**452**	680	**367**	551
×47	13.8	10.4	**413**	621	**338**	507
×42	12.4	9.30	**371**	558	**302**	453
×38	11.2	8.40	**335**	504	**273**	410
×34	10.0	7.50	**299**	450	**244**	366
WT12×31	9.11	6.83	**273**	410	**222**	333
×27.5	8.10	6.08	**243**	365	**198**	296
WT10.5×100.5	29.6	22.2	**886**	1330	**722**	1080
×91	26.8	20.1	**802**	1210	**653**	980
×83	24.4	18.3	**731**	1100	**595**	892
×73.5	21.6	16.2	**647**	972	**527**	790
×66	19.4	14.6	**581**	873	**475**	712
×61	17.9	13.4	**536**	805	**436**	653
×55.5	16.3	12.2	**488**	734	**397**	595
×50.5	14.9	11.2	**446**	670	**364**	546

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

WT10.5–WT9

Table 5-3 (continued)
Available Strength in Axial Tension
WT Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips ϕ_tP_n	Rupture kips P_n/Ω_t	Rupture kips ϕ_tP_n
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
WT10.5×46.5	13.7	10.3	**410**	617	**335**	502
×41.5	12.2	9.15	**365**	549	**297**	446
×36.5	10.7	8.02	**320**	481	**261**	391
×34	10.0	7.50	**299**	450	**244**	366
×31	9.13	6.85	**273**	411	**223**	334
×27.5	8.10	6.08	**243**	365	**198**	296
×24	7.07	5.30	**212**	318	**172**	258
WT10.5×28.5	8.37	6.28	**251**	377	**204**	306
×25	7.36	5.52	**220**	331	**179**	269
×22	6.49	4.87	**194**	292	**158**	237
WT9×155.5^h	45.8	34.4	**1370**	2060	**1120**	1680
×141.5^h	41.6	31.2	**1250**	1870	**1010**	1520
×129^h	37.9	28.4	**1130**	1710	**923**	1380
×117^h	34.4	25.8	**1030**	1550	**839**	1260
×105.5	31.1	23.3	**931**	1400	**757**	1140
×96	28.2	21.2	**844**	1270	**689**	1030
×87.5	25.7	19.3	**769**	1160	**627**	941
×79	23.2	17.4	**695**	1040	**566**	848
×71.5	21.0	15.8	**629**	945	**514**	770
×65	19.1	14.3	**572**	860	**465**	697
×59.5	17.5	13.1	**524**	788	**426**	639
×53	15.6	11.7	**467**	702	**380**	570
×48.5	14.3	10.7	**428**	644	**348**	522
×43	12.7	9.52	**380**	572	**309**	464
×38	11.2	8.40	**335**	504	**273**	410
WT9×35.5	10.4	7.80	**311**	468	**254**	380
×32.5	9.55	7.16	**286**	430	**233**	349
×30	8.82	6.62	**264**	397	**215**	323
×27.5	8.10	6.08	**243**	365	**198**	296
×25	7.33	5.50	**219**	330	**179**	268
WT9×23	6.77	5.08	**203**	305	**165**	248
×20	5.88	4.41	**176**	265	**143**	215
×17.5	5.15	3.86	**154**	232	**125**	188

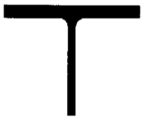
Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

^{h} Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-3 (continued)
Available Strength in Axial Tension
WT Shapes



WT8–WT7

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
WT8×50	14.7	11.0	**440**	662	**358**	536
×44.5	13.1	9.83	**392**	590	**319**	479
×38.5	11.3	8.48	**338**	509	**276**	413
×33.5	9.84	7.38	**295**	443	**240**	360
WT8×28.5	8.39	6.29	**251**	378	**204**	307
×25	7.37	5.53	**221**	332	**180**	270
×22.5	6.63	4.97	**199**	298	**162**	242
×20	5.89	4.42	**176**	265	**144**	215
×18	5.29	3.97	**158**	238	**129**	194
WT8×15.5	4.56	3.42	**137**	205	**111**	167
×13	3.84	2.88	**115**	173	**93.6**	140
WT7×365^h	107	80.3	**3200**	4820	**2610**	3910
×332.5^h	97.8	73.3	**2930**	4400	**2380**	3570
×302.5^h	88.9	66.7	**2660**	4000	**2170**	3250
×275^h	80.9	60.7	**2420**	3640	**1970**	2960
×250^h	73.5	55.1	**2200**	3310	**1790**	2690
×227.5^h	66.9	50.2	**2000**	3010	**1630**	2450
×213^h	62.6	47.0	**1870**	2820	**1530**	2290
×199^h	58.5	43.9	**1750**	2630	**1430**	2140
×185^h	54.4	40.8	**1630**	2450	**1330**	1990
×171^h	50.3	37.7	**1510**	2260	**1230**	1840
×155.5^h	45.7	34.3	**1370**	2060	**1110**	1670
×141.5^h	41.6	31.2	**1250**	1870	**1010**	1520
×128.5	37.8	28.4	**1130**	1700	**923**	1380
×116.5	34.2	25.7	**1020**	1540	**835**	1250
×105.5	31.0	23.3	**928**	1400	**757**	1140
×96.5	28.4	21.3	**850**	1280	**692**	1040
×88	25.9	19.4	**775**	1170	**631**	946
×79.5	23.4	17.6	**701**	1050	**572**	858
×72.5	21.3	16.0	**638**	959	**520**	780

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

WT7–WT6

Table 5-3 (continued)
Available Strength in Axial Tension
WT Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT7×66	19.4	14.6	581	873	475	712
×60	17.7	13.3	530	797	432	648
×54.5	16.0	12.0	479	720	390	585
×49.5	14.6	10.9	437	657	354	531
×45	13.2	9.90	395	594	322	483
WT7×41	12.0	9.00	359	540	293	439
×37	10.9	8.18	326	491	266	399
×34	9.99	7.49	299	450	243	365
×30.5	8.96	6.72	268	403	218	328
WT7×26.5	7.80	5.85	234	351	190	285
×24	7.07	5.30	212	318	172	258
×21.5	6.31	4.73	189	284	154	231
WT7×19	5.58	4.19	167	251	136	204
×17	5.00	3.75	150	225	122	183
×15	4.42	3.32	132	199	108	162
WT7×13	3.85	2.89	115	173	93.9	141
×11	3.25	2.44	97.3	146	79.3	119
WT6×168[h]	49.4	37.1	1480	2220	1210	1810
×152.5[h]	44.8	33.6	1340	2020	1090	1640
×139.5[h]	41.0	30.8	1230	1850	1000	1500
×126[h]	37.0	27.8	1110	1670	904	1360
×115[h]	33.9	25.4	1010	1530	826	1240
×105	30.9	23.2	925	1390	754	1130
×95	27.9	20.9	835	1260	679	1020
×85	25.0	18.8	749	1130	611	917
×76	22.4	16.8	671	1010	546	819
×68	20.0	15.0	599	900	488	731
×60	17.6	13.2	527	792	429	644
×53	15.6	11.7	467	702	380	570
×48	14.1	10.6	422	634	345	517
×43.5	12.8	9.60	383	576	312	468
×39.5	11.6	8.70	347	522	283	424
×36	10.6	7.95	317	477	258	388
×32.5	9.54	7.15	286	429	232	349

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

[h] Flange thickness is greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 50 ksi
F_u = 65 ksi

Table 5-3 (continued)
Available Strength in Axial Tension
WT Shapes

WT6–WT5

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
WT6×29	8.52	6.39	255	383	208	312
×26.5	7.78	5.84	233	350	190	285
WT6×25	7.30	5.48	219	329	178	267
×22.5	6.56	4.92	196	295	160	240
×20	5.84	4.38	175	263	142	214
WT6×17.5	5.17	3.88	155	233	126	189
×15	4.40	3.30	132	198	107	161
×13	3.82	2.87	114	172	93.3	140
WT6×11	3.24	2.43	97.0	146	79.0	118
×9.5	2.79	2.09	83.5	126	67.9	102
×8	2.36	1.77	70.7	106	57.5	86.3
×7	2.08	1.56	62.3	93.6	50.7	76.1
WT5×56	16.5	12.4	494	743	403	605
×50	14.7	11.0	440	662	358	536
×44	12.9	9.68	386	581	315	472
×38.5	11.3	8.48	338	509	276	413
×34	9.99	7.49	299	450	243	365
×30	8.82	6.62	264	397	215	323
×27	7.91	5.93	237	356	193	289
×24.5	7.21	5.41	216	324	176	264
WT5×22.5	6.63	4.97	199	298	162	242
×19.5	5.73	4.30	172	258	140	210
×16.5	4.85	3.64	145	218	118	177
WT5×15	4.42	3.32	132	199	108	162
×13	3.81	2.86	114	171	93.0	139
×11	3.24	2.43	97.0	146	79.0	118
WT5×9.5	2.81	2.11	84.1	126	68.6	103
×8.5	2.50	1.88	74.9	113	61.1	91.6
×7.5	2.21	1.66	66.2	99.5	54.0	80.9
×6	1.77	1.33	53.0	79.7	43.2	64.8

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

WT4

Table 5–3 (continued)
Available Strength in Axial Tension
WT Shapes

F_y = 50 ksi
F_u = 65 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
WT4×33.5	9.84	7.38	**295**	443	**240**	360
×29	8.54	6.41	**256**	384	**208**	312
×24	7.05	5.29	**211**	317	**172**	258
×20	5.87	4.40	**176**	264	**143**	215
×17.5	5.14	3.86	**154**	231	**125**	188
×15.5	4.56	3.42	**137**	205	**111**	167
WT4×14	4.12	3.09	**123**	185	**100**	151
×12	3.54	2.66	**106**	159	**86.5**	130
WT4×10.5	3.08	2.31	**92.2**	139	**75.1**	113
×9	2.63	1.97	**78.7**	118	**64.0**	96.0
WT4×7.5	2.22	1.67	**66.5**	99.9	**54.3**	81.4
×6.5	1.92	1.44	**57.5**	86.4	**46.8**	70.2
×5	1.48	1.11	**44.3**	66.6	**36.1**	54.1

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.923A_g$.

F_y = 46 ksi
F_u = 58 ksi

Table 5–4
Available Strength in Axial Tension
Rectangular HSS

HSS20–HSS16

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS20×12×5/8	35.0	26.3	**964**	1450	**763**	1140
×1/2	28.3	21.2	**780**	1170	**615**	922
×3/8	21.5	16.1	**592**	890	**467**	700
×5/16	18.1	13.6	**499**	749	**394**	592
HSS20×8×5/8	30.3	22.7	**835**	1250	**658**	987
×1/2	24.6	18.5	**678**	1020	**537**	805
×3/8	18.7	14.0	**515**	774	**406**	609
×5/16	15.7	11.8	**432**	650	**342**	513
HSS20×4×1/2	20.9	15.7	**576**	865	**455**	683
×3/8	16.0	12.0	**441**	662	**348**	522
×5/16	13.4	10.1	**369**	555	**293**	439
×1/4	10.8	8.10	**297**	447	**235**	352
HSS18×6×5/8	25.7	19.3	**708**	1060	**560**	840
×1/2	20.9	15.7	**576**	865	**455**	683
×3/8	16.0	12.0	**441**	662	**348**	522
×5/16	13.4	10.1	**369**	555	**293**	439
×1/4	10.8	8.10	**297**	447	**235**	352
HSS16×12×5/8	30.3	22.7	**835**	1250	**658**	987
×1/2	24.6	18.5	**678**	1020	**537**	805
×3/8	18.7	14.0	**515**	774	**406**	609
×5/16	15.7	11.8	**432**	650	**342**	513
HSS16×8×5/8	25.7	19.3	**708**	1060	**560**	840
×1/2	20.9	15.7	**576**	865	**455**	683
×3/8	16.0	12.0	**441**	662	**348**	522
×1/4	10.8	8.10	**297**	447	**235**	352
HSS16×4×5/8	21.0	15.8	**578**	869	**458**	687
×1/2	17.2	12.9	**474**	712	**374**	561
×3/8	13.2	9.90	**364**	546	**287**	431
×5/16	11.1	8.32	**306**	460	**241**	362
×1/4	8.96	6.72	**247**	371	**195**	292
×3/16	6.76	5.07	**186**	280	**147**	221

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

HSS14–HSS12

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

F_y = 46 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS14×10×5/8	25.7	19.3	**708**	1060	**560**	840
×1/2	20.9	15.7	**576**	865	**455**	683
×3/8	16.0	12.0	**441**	662	**348**	522
×5/16	13.4	10.1	**369**	555	**293**	439
×1/4	10.8	8.10	**297**	447	**235**	352
HSS14×6×5/8	21.0	15.8	**578**	869	**458**	687
×1/2	17.2	12.9	**474**	712	**374**	561
×3/8	13.2	9.90	**364**	546	**287**	431
×5/16	11.1	8.32	**306**	460	**241**	362
×1/4	8.96	6.72	**247**	371	**195**	292
×3/16	6.76	5.07	**186**	280	**147**	221
HSS14×4×5/8	18.7	14.0	**515**	774	**406**	609
×1/2	15.3	11.5	**421**	633	**334**	500
×3/8	11.8	8.85	**325**	489	**257**	385
×5/16	9.92	7.44	**273**	411	**216**	324
×1/4	8.03	6.02	**221**	332	**175**	262
×3/16	6.06	4.55	**167**	251	**132**	198
HSS12×10×1/2	19.0	14.3	**523**	787	**415**	622
×3/8	14.6	10.9	**402**	604	**316**	474
×5/16	12.2	9.15	**336**	505	**265**	398
×1/4	9.90	7.43	**273**	410	**215**	323
HSS12×8×5/8	21.0	15.8	**578**	869	**458**	687
×1/2	17.2	12.9	**474**	712	**374**	561
×3/8	13.2	9.90	**364**	546	**287**	431
×5/16	11.1	8.32	**306**	460	**241**	362
×1/4	8.96	6.72	**247**	371	**195**	292
×3/16	6.76	5.07	**186**	280	**147**	221
HSS12×6×5/8	18.7	14.0	**515**	774	**406**	609
×1/2	15.3	11.5	**421**	633	**334**	500
×3/8	11.8	8.85	**325**	489	**257**	385
×5/16	9.92	7.44	**273**	411	**216**	324
×1/4	8.03	6.02	**221**	332	**175**	262
×3/16	6.06	4.55	**167**	251	**132**	198

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

F_y = 46 ksi
F_u = 58 ksi

Table 5–4 (continued)
Available Strength in Axial Tension
Rectangular HSS

HSS12–HSS10

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips		Rupture kips	
			P_n/Ω_t	ϕ_tP_n	P_n/Ω_t	ϕ_tP_n
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS12×4×$^5/_8$	16.4	12.3	**452**	679	**357**	535
×$^1/_2$	13.5	10.1	**372**	559	**293**	439
×$^3/_8$	10.4	7.80	**286**	431	**226**	339
×$^5/_{16}$	8.76	6.57	**241**	363	**191**	286
×$^1/_4$	7.10	5.33	**196**	294	**155**	232
×$^3/_{16}$	5.37	4.03	**148**	222	**117**	175
HSS12×3$^1/_2$×$^3/_8$	10.0	7.50	**275**	414	**218**	326
×$^5/_{16}$	8.46	6.34	**233**	350	**184**	276
HSS12×3×$^5/_{16}$	8.17	6.13	**225**	338	**178**	267
×$^1/_4$	6.63	4.97	**183**	274	**144**	216
×$^3/_{16}$	5.02	3.76	**138**	208	**109**	164
HSS12×2×$^5/_{16}$	7.59	5.69	**209**	314	**165**	248
×$^1/_4$	6.17	4.63	**170**	255	**134**	201
×$^3/_{16}$	4.67	3.50	**129**	193	**102**	152
HSS10×8×$^5/_8$	18.7	14.0	**515**	774	**406**	609
×$^1/_2$	15.3	11.5	**421**	633	**334**	500
×$^3/_8$	11.8	8.85	**325**	489	**257**	385
×$^5/_{16}$	9.92	7.44	**273**	411	**216**	324
×$^1/_4$	8.03	6.02	**221**	332	**175**	262
×$^3/_{16}$	6.06	4.55	**167**	251	**132**	198
HSS10×6×$^5/_8$	16.4	12.3	**452**	679	**357**	535
×$^1/_2$	13.5	10.1	**372**	559	**293**	439
×$^3/_8$	10.4	7.80	**286**	431	**226**	339
×$^5/_{16}$	8.76	6.57	**241**	363	**191**	286
×$^1/_4$	7.10	5.33	**196**	294	**155**	232
×$^3/_{16}$	5.37	4.03	**148**	222	**117**	175
HSS10×5×$^3/_8$	9.67	7.25	**266**	400	**210**	315
×$^5/_{16}$	8.17	6.13	**225**	338	**178**	267
×$^1/_4$	6.63	4.97	**183**	274	**144**	216
×$^3/_{16}$	5.02	3.76	**138**	208	**109**	164

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

HSS10–HSS9

Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

F_y = 46 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS10×4×5/8	14.0	10.5	**386**	580	**305**	457
×1/2	11.6	8.70	**320**	480	**252**	378
×3/8	8.97	6.73	**247**	371	**195**	293
×5/16	7.59	5.69	**209**	314	**165**	248
×1/4	6.17	4.63	**170**	255	**134**	201
×3/16	4.67	3.50	**129**	193	**102**	152
×1/8	3.16	2.37	**87.0**	131	**68.7**	103
HSS10×3 1/2×1/2	11.1	8.32	**306**	460	**241**	362
×3/8	8.62	6.47	**237**	357	**188**	281
×5/16	7.30	5.48	**201**	302	**159**	238
×1/4	5.93	4.45	**163**	246	**129**	194
×3/16	4.50	3.38	**124**	186	**98.0**	147
×1/8	3.04	2.28	**83.7**	126	**66.1**	99.2
HSS10×3×3/8	8.27	6.20	**228**	342	**180**	270
×5/16	7.01	5.26	**193**	290	**153**	229
×1/4	5.70	4.27	**157**	236	**124**	186
×3/16	4.32	3.24	**119**	179	**94.0**	141
×1/8	2.93	2.20	**80.7**	121	**63.8**	95.7
HSS10×2×3/8	7.58	5.69	**209**	314	**165**	248
×5/16	6.43	4.82	**177**	266	**140**	210
×1/4	5.24	3.93	**144**	217	**114**	171
×3/16	3.98	2.99	**110**	165	**86.7**	130
×1/8	2.70	2.03	**74.4**	112	**58.9**	88.3
HSS9×7×5/8	16.4	12.3	**452**	679	**357**	535
×1/2	13.5	10.1	**372**	559	**293**	439
×3/8	10.4	7.80	**286**	431	**226**	339
×5/16	8.76	6.57	**241**	363	**191**	286
×1/4	7.10	5.33	**196**	294	**155**	232
×3/16	5.37	4.03	**148**	222	**117**	175

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

F_y = 46 ksi
F_u = 58 ksi

Table 5–4 (continued)
Available Strength in Axial Tension
Rectangular HSS

HSS9–HSS8

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	**ASD**	LRFD	**ASD**	LRFD
HSS9×5×5/8	14.0	10.5	**386**	580	**305**	457
×1/2	11.6	8.70	**320**	480	**252**	378
×3/8	8.97	6.73	**247**	371	**195**	293
×5/16	7.59	5.69	**209**	314	**165**	248
×1/4	6.17	4.63	**170**	255	**134**	201
×3/16	4.67	3.50	**129**	193	**102**	152
HSS9×3×1/2	9.74	7.30	**268**	403	**212**	318
×3/8	7.58	5.69	**209**	314	**165**	248
×5/16	6.43	4.82	**177**	266	**140**	210
×1/4	5.24	3.93	**144**	217	**114**	171
×3/16	3.98	2.99	**110**	165	**86.7**	130
HSS8×6×5/8	14.0	10.5	**386**	580	**305**	457
×1/2	11.6	8.70	**320**	480	**252**	378
×3/8	8.97	6.73	**247**	371	**195**	293
×5/16	7.59	5.69	**209**	314	**165**	248
×1/4	6.17	4.63	**170**	255	**134**	201
×3/16	4.67	3.50	**129**	193	**102**	152
HSS8×4×5/8	11.7	8.78	**322**	484	**255**	382
×1/2	9.74	7.30	**268**	403	**212**	318
×3/8	7.58	5.69	**209**	314	**165**	248
×5/16	6.43	4.82	**177**	266	**140**	210
×1/4	5.24	3.93	**144**	217	**114**	171
×3/16	3.98	2.99	**110**	165	**86.7**	130
×1/8	2.70	2.03	**74.4**	112	**58.9**	88.3
HSS8×3×1/2	8.81	6.61	**243**	365	**192**	288
×3/8	6.88	5.16	**190**	285	**150**	224
×5/16	5.85	4.39	**161**	242	**127**	191
×1/4	4.77	3.58	**131**	197	**104**	156
×3/16	3.63	2.72	**100**	150	**78.9**	118
×1/8	2.46	1.85	**67.8**	102	**53.7**	80.5

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

HSS8–HSS6

Table 5–4 (continued)
Available Strength in Axial Tension
Rectangular HSS

F_y = 46 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS8×2×3/8	6.18	4.63	170	256	134	201
×5/16	5.26	3.94	145	218	114	171
×1/4	4.30	3.22	118	178	93.4	140
×3/16	3.28	2.46	90.3	136	71.3	107
×1/8	2.23	1.67	61.4	92.3	48.4	72.6
HSS7×5×1/2	9.74	7.30	268	403	212	318
×3/8	7.58	5.69	209	314	165	248
×5/16	6.43	4.82	177	266	140	210
×1/4	5.24	3.93	144	217	114	171
×3/16	3.98	2.99	110	165	86.7	130
×1/8	2.70	2.03	74.4	112	58.9	88.3
HSS7×4×1/2	8.81	6.61	243	365	192	288
×3/8	6.88	5.16	190	285	150	224
×5/16	5.85	4.39	161	242	127	191
×1/4	4.77	3.58	131	197	104	156
×3/16	3.63	2.72	100	150	78.9	118
×1/8	2.46	1.85	67.8	102	53.7	80.5
HSS7×3×1/2	7.88	5.91	217	326	171	257
×3/8	6.18	4.63	170	256	134	201
×5/16	5.26	3.94	145	218	114	171
×1/4	4.30	3.22	118	178	93.4	140
×3/16	3.28	2.46	90.3	136	71.3	107
×1/8	2.23	1.67	61.4	92.3	48.4	72.6
HSS7×2×1/4	3.84	2.88	106	159	83.5	125
×3/16	2.93	2.20	80.7	121	63.8	95.7
×1/8	2.00	1.50	55.1	82.8	43.5	65.3
HSS6×5×1/2	8.81	6.61	243	365	192	288
×3/8	6.88	5.16	190	285	150	224
×5/16	5.85	4.39	161	242	127	191
×1/4	4.77	3.58	131	197	104	156
×3/16	3.63	2.72	100	150	78.9	118
×1/8	2.46	1.85	67.8	102	53.7	80.5

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

F_y = 46 ksi
F_u = 58 ksi

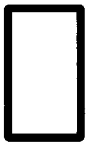
Table 5-4 (continued)
Available Strength in Axial Tension
Rectangular HSS

HSS6–HSS5

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips P_n/Ω_t	Yielding kips $\phi_t P_n$	Rupture kips P_n/Ω_t	Rupture kips $\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS6×4×1/2	7.88	5.91	217	326	171	257
×3/8	6.18	4.63	170	256	134	201
×5/16	5.26	3.94	145	218	114	171
×1/4	4.30	3.22	118	178	93.4	140
×3/16	3.28	2.46	90.3	136	71.3	107
×1/8	2.23	1.67	61.4	92.3	48.4	72.6
HSS6×3×1/2	6.95	5.21	191	288	151	227
×3/8	5.48	4.11	151	227	119	179
×5/16	4.68	3.51	129	194	102	153
×1/4	3.84	2.88	106	159	83.5	125
×3/16	2.93	2.20	80.7	121	63.8	95.7
×1/8	2.00	1.50	55.1	82.8	43.5	65.3
HSS6×2×3/8	4.78	3.58	132	198	104	156
×5/16	4.10	3.08	113	170	89.3	134
×1/4	3.37	2.53	92.8	140	73.4	110
×3/16	2.58	1.94	71.1	107	56.3	84.4
×1/8	1.77	1.33	48.8	73.3	38.6	57.9
HSS5×4×1/2	6.95	5.21	191	288	151	227
×3/8	5.48	4.11	151	227	119	179
×5/16	4.68	3.51	129	194	102	153
×1/4	3.84	2.88	106	159	83.5	125
×3/16	2.93	2.20	80.7	121	63.8	95.7
×1/8	2.00	1.50	55.1	82.8	43.5	65.3
HSS5×3×1/2	6.02	4.51	166	249	131	196
×3/8	4.78	3.58	132	198	104	156
×5/16	4.10	3.08	113	170	89.3	134
×1/4	3.37	2.53	92.8	140	73.4	110
×3/16	2.58	1.94	71.1	107	56.3	84.4
×1/8	1.77	1.33	48.8	73.3	38.6	57.9
HSS5×2 1/2×1/4	3.14	2.36	86.5	130	68.4	103
×3/16	2.41	1.81	66.4	99.8	52.5	78.7
×1/8	1.65	1.24	45.4	68.3	36.0	53.9

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.



HSS5–HSS3½

Table 5–4 (continued)
Available Strength in Axial Tension
Rectangular HSS

F_y = 46 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS5×2×3/8	4.09	3.07	**113**	169	**89.0**	134
×5/16	3.52	2.64	**97.0**	146	**76.6**	115
×1/4	2.91	2.18	**80.2**	120	**63.2**	94.8
×3/16	2.24	1.68	**61.7**	92.7	**48.7**	73.1
×1/8	1.54	1.16	**42.4**	63.8	**33.6**	50.5
HSS4×3×3/8	4.09	3.07	**113**	169	**89.0**	134
×5/16	3.52	2.64	**97.0**	146	**76.6**	115
×1/4	2.91	2.18	**80.2**	120	**63.2**	94.8
×3/16	2.24	1.68	**61.7**	92.7	**48.7**	73.1
×1/8	1.54	1.16	**42.4**	63.8	**33.6**	50.5
HSS4×2½×3/8	3.74	2.81	**103**	155	**81.5**	122
×5/16	3.23	2.42	**89.0**	134	**70.2**	105
×1/4	2.67	2.00	**73.5**	111	**58.0**	87.0
×3/16	2.06	1.55	**56.7**	85.3	**45.0**	67.4
×1/8	1.42	1.07	**39.1**	58.8	**31.0**	46.5
HSS4×2×3/8	3.39	2.54	**93.4**	140	**73.7**	110
×5/16	2.94	2.21	**81.0**	122	**64.1**	96.1
×1/4	2.44	1.83	**67.2**	101	**53.1**	79.6
×3/16	1.89	1.42	**52.1**	78.2	**41.2**	61.8
×1/8	1.30	0.975	**35.8**	53.8	**28.3**	42.4
HSS3½×2½×3/8	3.39	2.54	**93.4**	140	**73.7**	110
×5/16	2.94	2.21	**81.0**	122	**64.1**	96.1
×1/4	2.44	1.83	**67.2**	101	**53.1**	79.6
×3/16	1.89	1.42	**52.1**	78.2	**41.2**	61.8
×1/8	1.30	0.975	**35.8**	53.8	**28.3**	42.4
HSS3½×2×1/4	2.21	1.66	**60.9**	91.5	**48.1**	72.2
×3/16	1.71	1.28	**47.1**	70.8	**37.1**	55.7
×1/8	1.19	0.892	**32.8**	49.3	**25.9**	38.8
×1/4	1.97	1.48	**54.3**	81.6	**42.9**	64.4
×3/16	1.54	1.16	**42.4**	63.8	**33.6**	50.5
×1/8	1.07	0.803	**29.5**	44.3	**23.3**	34.9

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

F_y = 46 ksi
F_u = 58 ksi

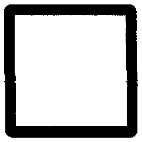
Table 5–4 (continued)
Available Strength in Axial Tension
Rectangular HSS

HSS3–HSS2

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS3×2½×5/16	2.64	1.98	**72.7**	109	**57.4**	86.1
×1/4	2.21	1.66	**60.9**	91.5	**48.1**	72.2
×3/16	1.71	1.28	**47.1**	70.8	**37.1**	55.7
×1/8	1.19	0.892	**32.8**	49.3	**25.9**	38.8
HSS3×2×5/16	2.35	1.76	**64.7**	97.3	**51.0**	76.6
×1/4	1.97	1.48	**54.3**	81.6	**42.9**	64.4
×3/16	1.54	1.16	**42.4**	63.8	**33.6**	50.5
×1/8	1.07	0.803	**29.5**	44.3	**23.3**	34.9
HSS3×1½×1/4	1.74	1.30	**47.9**	72.0	**37.7**	56.6
×3/16	1.37	1.03	**37.7**	56.7	**29.9**	44.8
HSS3×1×1/8	0.956	0.717	**26.3**	39.6	**20.8**	31.2
×3/16	1.19	0.892	**32.8**	49.3	**25.9**	38.8
×1/8	0.840	0.630	**23.1**	34.8	**18.3**	27.4
HSS2½×2×1/4	1.74	1.30	**47.9**	72.0	**37.7**	56.6
×3/16	1.37	1.03	**37.7**	56.7	**29.9**	44.8
×1/8	0.956	0.717	**26.3**	39.6	**20.8**	31.2
HSS2½×1½×1/4	1.51	1.13	**41.6**	62.5	**32.8**	49.2
×3/16	1.19	0.892	**32.8**	49.3	**25.9**	38.8
×1/8	0.840	0.630	**23.1**	34.8	**18.3**	27.4
HSS2½×1×3/16	1.02	0.765	**28.1**	42.2	**22.2**	33.3
×1/8	0.724	0.543	**19.9**	30.0	**15.7**	23.6
HSS2¼×2×3/16	1.28	0.960	**35.3**	53.0	**27.8**	41.8
×1/8	0.898	0.674	**24.7**	37.2	**19.5**	29.3
HSS2×1½×3/16	1.02	0.765	**28.1**	42.2	**22.2**	33.3
×1/8	0.724	0.543	**19.9**	30.0	**15.7**	23.6
HSS2×1×3/16	0.845	0.634	**23.3**	35.0	**18.4**	27.6
×1/8	0.608	0.456	**16.7**	25.2	**13.2**	19.8

Limit State	ASD	LRFD
Yielding	Ω_t = 1.67	ϕ_t = 0.90
Rupture	Ω_t = 2.00	ϕ_t = 0.75

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.



HSS16–HSS8

Table 5–5

Available Strength in Axial Tension

Square HSS

F_y = 46 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS16×16×5/8	35.0	26.3	**964**	1450	**763**	1140
×1/2	28.3	21.2	**780**	1170	**615**	922
×3/8	21.5	16.1	**592**	890	**467**	700
×5/16	18.1	13.6	**499**	749	**394**	592
HSS14×14×5/8	30.3	22.7	**835**	1250	**658**	987
×1/2	24.6	18.5	**678**	1020	**537**	805
×3/8	18.7	14.0	**515**	774	**406**	609
×5/16	15.7	11.8	**432**	650	**342**	513
HSS12×12×5/8	25.7	19.3	**708**	1060	**560**	840
×1/2	20.9	15.7	**576**	865	**455**	683
×3/8	16.0	12.0	**441**	662	**348**	522
×5/16	13.4	10.1	**369**	555	**293**	439
×1/4	10.8	8.10	**297**	447	**235**	352
×3/16	8.15	6.11	**224**	337	**177**	266
HSS10×10×5/8	21.0	15.8	**578**	869	**458**	687
×1/2	17.2	12.9	**474**	712	**374**	561
×3/8	13.2	9.90	**364**	546	**287**	431
×5/16	11.1	8.32	**306**	460	**241**	362
×1/4	8.96	6.72	**247**	371	**195**	292
×3/16	6.76	5.07	**186**	280	**147**	221
HSS9×9×5/8	18.7	14.0	**515**	774	**406**	609
×1/2	15.3	11.5	**421**	633	**334**	500
×3/8	11.8	8.85	**325**	489	**257**	385
×5/16	9.92	7.44	**273**	411	**216**	324
×1/4	8.03	6.02	**221**	332	**175**	262
×3/16	6.06	4.55	**167**	251	**132**	198
×1/8	4.09	3.07	**113**	169	**89.0**	134
HSS8×8×5/8	16.4	12.3	**452**	679	**357**	535
×1/2	13.5	10.1	**372**	559	**293**	439
×3/8	10.4	7.80	**286**	431	**226**	339
×5/16	8.76	6.57	**241**	363	**191**	286
×1/4	7.10	5.33	**196**	294	**155**	232
×3/16	5.37	4.03	**148**	222	**117**	175
×1/8	3.62	2.71	**99.7**	150	**78.6**	118

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

F_y = 46 ksi
F_u = 58 ksi

Table 5-5 (continued)
Available Strength in Axial Tension
Square HSS

HSS7–HSS4½

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
HSS7×7×5/8	14.0	10.5	386	580	305	457
×1/2	11.6	8.70	320	480	252	378
×3/8	8.97	6.73	247	371	195	293
×5/16	7.59	5.69	209	314	165	248
×1/4	6.17	4.63	170	255	134	201
×3/16	4.67	3.50	129	193	102	152
×1/8	3.16	2.37	87.0	131	68.7	103
HSS6×6×5/8	11.7	8.78	322	484	255	382
×1/2	9.74	7.30	268	403	212	318
×3/8	7.58	5.69	209	314	165	248
×5/16	6.43	4.82	177	266	140	210
×1/4	5.24	3.93	144	217	114	171
×3/16	3.98	2.99	110	165	86.7	130
×1/8	2.70	2.03	74.4	112	58.9	88.3
HSS5½×5½×3/8	6.88	5.16	190	285	150	224
×5/16	5.85	4.39	161	242	127	191
×1/4	4.77	3.58	131	197	104	156
×3/16	3.63	2.72	100	150	78.9	118
×1/8	2.46	1.85	67.8	102	53.7	80.5
HSS5×5×1/2	7.88	5.91	217	326	171	257
×3/8	6.18	4.63	170	256	134	201
×5/16	5.26	3.94	145	218	114	171
×1/4	4.30	3.22	118	178	93.4	140
×3/16	3.28	2.46	90.3	136	71.3	107
×1/8	2.23	1.67	61.4	92.3	48.4	72.6
HSS4½×4½×1/2	6.95	5.21	191	288	151	227
×3/8	5.48	4.11	151	227	119	179
×5/16	4.68	3.51	129	194	102	153
×1/4	3.84	2.88	106	159	83.5	125
×3/16	2.93	2.20	80.7	121	63.8	95.7
×1/8	2.00	1.50	55.1	82.8	43.5	65.3

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

HSS4–HSS2

Table 5–5 (continued)
Available Strength in Axial Tension
Square HSS

F_y = 46 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	**ASD**	LRFD	**ASD**	LRFD
HSS4×4×1/2	6.02	4.51	**166**	249	**131**	196
×3/8	4.78	3.58	**132**	198	**104**	156
×5/16	4.10	3.08	**113**	170	**89.3**	134
×1/4	3.37	2.53	**92.8**	140	**73.4**	110
×3/16	2.58	1.94	**71.1**	107	**56.3**	84.4
×1/8	1.77	1.33	**48.8**	73.3	**38.6**	57.9
HSS3½×3½×3/8	4.09	3.07	**113**	169	**89.0**	134
×5/16	3.52	2.64	**97.0**	146	**76.6**	115
×1/4	2.91	2.18	**80.2**	120	**63.2**	94.8
×3/16	2.24	1.68	**61.7**	92.7	**48.7**	73.1
×1/8	1.54	1.16	**42.4**	63.8	**33.6**	50.5
HSS3×3×3/8	3.39	2.54	**93.4**	140	**73.7**	110
×5/16	2.94	2.21	**81.0**	122	**64.1**	96.1
×1/4	2.44	1.83	**67.2**	101	**53.1**	79.6
×3/16	1.89	1.42	**52.1**	78.2	**41.2**	61.8
×1/8	1.30	0.975	**35.8**	53.8	**28.3**	42.4
HSS2½×2½×5/16	2.35	1.76	**64.7**	97.3	**51.0**	76.6
×1/4	1.97	1.48	**54.3**	81.6	**42.9**	64.4
×3/16	1.54	1.16	**42.4**	63.8	**33.6**	50.5
×1/8	1.07	0.803	**29.5**	44.3	**23.3**	34.9
HSS2¼×2¼×1/4	1.74	1.30	**47.9**	72.0	**37.7**	56.6
×3/16	1.37	1.03	**37.7**	56.7	**29.9**	44.8
×1/8	0.956	0.717	**26.3**	39.6	**20.8**	31.2
HSS2×2×1/4	1.51	1.13	**41.6**	62.5	**32.8**	49.2
×3/16	1.19	0.892	**32.8**	49.3	**25.9**	38.8
×1/8	0.840	0.630	**23.1**	34.8	**18.3**	27.4

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.952A_g$.

F_y = 42 ksi
F_u = 58 ksi

Table 5–6
Available Strength in Axial Tension
Round HSS

HSS20.000–
HSS10.000

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	**ASD**	LRFD	**ASD**	LRFD
HSS20.000×0.375	21.5	16.1	**541**	813	**467**	700
HSS18.000×0.500	25.6	19.2	**644**	968	**557**	835
×0.375	19.4	14.6	**488**	733	**423**	635
HSS16.000×0.625	28.1	21.1	**707**	1060	**612**	918
×0.500	22.7	17.0	**571**	858	**493**	740
×0.438	19.9	14.9	**500**	752	**432**	648
×0.375	17.2	12.9	**433**	650	**374**	561
×0.312	14.4	10.8	**362**	544	**313**	470
×0.250	11.5	8.63	**289**	435	**250**	375
HSS14.000×0.625	24.5	18.4	**616**	926	**534**	800
×0.500	19.8	14.9	**498**	748	**432**	648
×0.375	15.0	11.3	**377**	567	**328**	492
×0.312	12.5	9.38	**314**	473	**272**	408
×0.250	10.1	7.58	**254**	382	**220**	330
HSS12.750×0.500	17.9	13.4	**450**	677	**389**	583
×0.375	13.6	10.2	**342**	514	**296**	444
×0.250	9.16	6.87	**230**	346	**199**	299
HSS10.750×0.500	15.0	11.3	**377**	567	**328**	492
×0.375	11.4	8.55	**287**	431	**248**	372
×0.250	7.70	5.78	**194**	291	**168**	251
HSS10.000×0.625	17.2	12.9	**433**	650	**374**	561
×0.500	13.9	10.4	**350**	525	**302**	452
×0.375	10.6	7.95	**267**	401	**231**	346
×0.312	8.88	6.66	**223**	336	**193**	290
×0.250	7.15	5.36	**180**	270	**155**	233
×0.188	5.37	4.03	**135**	203	**117**	175

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.869A_g$.

HSS9.625–
HSS6.875

Table 5–6 (continued)
Available Strength in Axial Tension
Round HSS

F_y = 42 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS9.625×0.500	13.4	10.1	337	507	293	439
×0.375	10.2	7.65	257	386	222	333
×0.312	8.53	6.40	215	322	186	278
×0.250	6.87	5.15	173	260	149	224
×0.188	5.17	3.88	130	195	113	169
HSS8.625×0.625	14.7	11.0	370	556	319	479
×0.500	11.9	8.92	299	450	259	388
×0.375	9.07	6.80	228	343	197	296
×0.322	7.85	5.89	197	297	171	256
×0.250	6.14	4.60	154	232	133	200
×0.188	4.62	3.47	116	175	101	151
HSS7.625×0.375	7.98	5.99	201	302	174	261
×0.328	7.01	5.26	176	265	153	229
HSS7.500×0.500	10.3	7.73	259	389	224	336
×0.375	7.84	5.88	197	296	171	256
×0.312	6.59	4.94	166	249	143	215
×0.250	5.32	3.99	134	201	116	174
×0.188	4.00	3.00	101	151	87.0	131
HSS7.000×0.500	9.55	7.16	240	361	208	311
×0.375	7.29	5.47	183	276	159	238
×0.312	6.13	4.60	154	232	133	200
×0.250	4.95	3.71	124	187	108	161
×0.188	3.73	2.80	93.8	141	81.2	122
×0.125	2.51	1.88	63.1	94.9	54.5	81.8
HSS6.875×0.500	9.36	7.02	235	354	204	305
×0.375	7.16	5.37	180	271	156	234
×0.312	6.02	4.51	151	228	131	196
×0.250	4.86	3.64	122	184	106	158
×0.188	3.66	2.75	92.0	138	79.8	120

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.869A_g$.

F_y = 42 ksi
F_u = 58 ksi

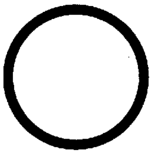
Table 5–6 (continued)
Available Strength in Axial Tension
Round HSS

HSS6.625–
HSS5.000

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS6.625×0.500	9.00	6.75	226	340	196	294
×0.432	7.86	5.90	198	297	171	257
×0.375	6.88	5.16	173	260	150	224
×0.312	5.79	4.34	146	219	126	189
×0.280	5.20	3.90	131	197	113	170
×0.250	4.68	3.51	118	177	102	153
×0.188	3.53	2.65	88.8	133	76.9	115
×0.125	2.37	1.78	59.6	89.6	51.6	77.4
HSS6.000×0.500	8.09	6.07	203	306	176	264
×0.375	6.20	4.65	156	234	135	202
×0.312	5.22	3.92	131	197	114	171
×0.280	4.69	3.52	118	177	102	153
×0.250	4.22	3.17	106	160	91.9	138
×0.188	3.18	2.39	80.0	120	69.3	104
×0.125	2.14	1.61	53.8	80.9	46.7	70.0
HSS5.563×0.500	7.45	5.59	187	282	162	243
×0.375	5.72	4.29	144	216	124	187
×0.258	4.01	3.01	101	152	87.3	131
×0.188	2.95	2.21	74.2	112	64.1	96.1
×0.134	2.12	1.59	53.3	80.1	46.1	69.2
HSS5.500×0.500	7.36	5.52	185	278	160	240
×0.375	5.65	4.24	142	214	123	184
×0.258	3.97	2.98	99.8	150	86.4	130
×0.500	6.62	4.97	166	250	144	216
HSS5.000×0.375	5.10	3.82	128	193	111	166
×0.312	4.30	3.22	108	163	93.4	140
×0.258	3.59	2.69	90.3	136	78.0	117
×0.250	3.49	2.62	87.8	132	76.0	114
×0.188	2.64	1.98	66.4	99.8	57.4	86.1
×0.125	1.78	1.34	44.8	67.3	38.9	58.3

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.869A_g$.



HSS4.500–
HSS2.500

Table 5–6 (continued)
Available Strength in Axial Tension
Round HSS

F_y = 42 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	ϕ_tP_n	P_n/Ω_t	ϕ_tP_n
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS4.500×0.375	4.55	3.41	**114**	172	**98.9**	148
×0.337	4.12	3.09	**104**	156	**89.6**	134
×0.237	2.96	2.22	**74.4**	112	**64.4**	96.6
×0.188	2.36	1.77	**59.4**	89.2	**51.3**	77.0
×0.125	1.60	1.20	**40.2**	60.5	**34.8**	52.2
HSS4.000×0.313	3.39	2.54	**85.3**	128	**73.7**	110
×0.250	2.76	2.07	**69.4**	104	**60.0**	90.0
×0.237	2.61	1.96	**65.6**	98.7	**56.8**	85.3
×0.226	2.50	1.88	**62.9**	94.5	**54.5**	81.8
×0.220	2.44	1.83	**61.4**	92.2	**53.1**	79.6
×0.188	2.09	1.57	**52.6**	79.0	**45.5**	68.3
×0.125	1.42	1.07	**35.7**	53.7	**31.0**	46.5
HSS3.500×0.313	2.93	2.20	**73.7**	111	**63.8**	95.7
×0.300	2.82	2.11	**70.9**	107	**61.2**	91.8
×0.250	2.39	1.79	**60.1**	90.3	**51.9**	77.9
×0.216	2.08	1.56	**52.3**	78.6	**45.2**	67.9
×0.203	1.97	1.48	**49.5**	74.5	**42.9**	64.4
×0.188	1.82	1.36	**45.8**	68.8	**39.4**	59.2
×0.125	1.23	0.923	**30.9**	46.5	**26.8**	40.2
HSS3.000×0.250	2.03	1.52	**51.1**	76.7	**44.1**	66.1
×0.216	1.77	1.33	**44.5**	66.9	**38.6**	57.9
×0.203	1.67	1.25	**42.0**	63.1	**36.3**	54.4
×0.188	1.54	1.16	**38.7**	58.2	**33.6**	50.5
×0.152	1.27	0.953	**31.9**	48.0	**27.6**	41.5
×0.134	1.12	0.840	**28.2**	42.3	**24.4**	36.5
×0.125	1.05	0.788	**26.4**	39.7	**22.9**	34.3
HSS2.875×0.250	1.93	1.45	**48.5**	73.0	**42.1**	63.1
×0.203	1.59	1.19	**40.0**	60.1	**34.5**	51.8
×0.188	1.48	1.11	**37.2**	55.9	**32.2**	48.3
×0.125	1.01	0.758	**25.4**	38.2	**22.0**	33.0
HSS2.500×0.250	1.66	1.25	**41.7**	62.7	**36.3**	54.4
×0.188	1.27	0.953	**31.9**	48.0	**27.6**	41.5
×0.125	0.869	0.652	**21.9**	32.8	**18.9**	28.4

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.869A_g$.

F_y = 42 ksi
F_u = 58 ksi

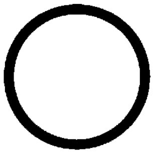
Table 5–6 (continued)
Available Strength in Axial Tension
Round HSS

HSS2.375–
HSS1.660

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	ϕ_tP_n	P_n/Ω_t	ϕ_tP_n
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
HSS2.375×0.250	1.57	1.18	**39.5**	59.3	**34.2**	51.3
×0.218	1.39	1.04	**35.0**	52.5	**30.2**	45.2
×0.188	1.20	0.900	**30.2**	45.4	**26.1**	39.1
×0.154	1.00	0.750	**25.1**	37.8	**21.8**	32.6
×0.125	0.823	0.617	**20.7**	31.1	**17.9**	26.8
HSS1.900×0.188	0.943	0.707	**23.7**	35.6	**20.5**	30.8
×0.145	0.749	0.562	**18.8**	28.3	**16.3**	24.4
×0.120	0.624	0.468	**15.7**	23.6	**13.6**	20.4
HSS1.660×0.140	0.625	0.469	**15.7**	23.6	**13.6**	20.4

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.869A_g$.



PIPE12–
PIPE1½

Table 5–7
Available Strength in Axial Tension
Pipe

F_y = 35 ksi
F_u = 60 ksi

Shape	Gross Area, A_g	A_e = 0.75A_g	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.²	in.²	ASD	LRFD	ASD	LRFD
Pipe 12 X-Strong	17.9	13.4	375	564	402	603
Std	13.6	10.2	285	428	306	459
Pipe 10 X-Strong	15.0	11.3	314	473	339	509
Std	11.1	8.32	233	350	250	374
Pipe 8 XX-Strong	20.0	15.0	419	630	450	675
X-Strong	11.9	8.92	249	375	268	401
Std	7.85	5.89	165	247	177	265
Pipe 6 XX-Strong	14.7	11.0	308	463	330	495
X-Strong	7.88	5.91	165	248	177	266
Std	5.22	3.92	109	164	118	176
Pipe 5 XX-Strong	10.7	8.02	224	337	241	361
X-Strong	5.72	4.29	120	180	129	193
Std	4.03	3.02	84.5	127	90.6	136
Pipe 4 XX-Strong	7.64	5.73	160	241	172	258
X-Strong	4.14	3.10	86.8	130	93.0	140
Std	2.97	2.23	62.2	93.6	66.9	100
Pipe 3½ X-Strong	3.44	2.58	72.1	108	77.4	116
Std	2.51	1.88	52.6	79.1	56.4	84.6
Pipe 3 XX-Strong	5.16	3.87	108	163	116	174
X-Strong	2.83	2.12	59.3	89.1	63.6	95.4
Std	2.08	1.56	43.6	65.5	46.8	70.2
Pipe 2½ XX-Strong	3.81	2.86	79.9	120	85.8	129
X-Strong	2.11	1.58	44.2	66.5	47.4	71.1
Std	1.59	1.19	33.3	50.1	35.7	53.6
Pipe 2 XX-Strong	2.51	1.88	52.6	79.1	56.4	84.6
X-Strong	1.39	1.04	29.1	43.8	31.2	46.8
Std	1.00	0.750	21.0	31.5	22.5	33.8
Pipe 1½ X-Strong	1.00	0.750	21.0	31.5	22.5	33.8
Std	0.750	0.563	15.7	23.6	16.9	25.3

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.700A_g$.

F_y = 35 ksi
F_u = 60 ksi

Table 5–7 (continued)
Available Strength in Axial Tension
Pipe

PIPE1¼–PIPE½

Shape	Gross Area, A_g	A_e = $0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
Pipe 1¼ X-Strong	0.830	0.623	17.4	26.1	18.7	28.0
Std	0.620	0.465	13.0	19.5	14.0	20.9
Pipe 1 X-Strong	0.600	0.450	12.6	18.9	13.5	20.3
Std	0.460	0.345	9.64	14.5	10.4	15.5
Pipe ¾ X-Strong	0.410	0.308	8.59	12.9	9.24	13.9
Std	0.310	0.232	6.50	9.77	6.96	10.4
Pipe ½ X-Strong	0.300	0.225	6.29	9.45	6.75	10.1
Std	0.230	0.173	4.82	7.24	5.19	7.79

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.700A_g$.

2L8–2L6

Table 5–8
Available Strength in Axial Tension
Double Angles

$F_y = 36$ ksi
$F_u = 58$ ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	**ASD**	LRFD	**ASD**	LRFD
2L8×8×1$^1/_8$	33.4	25	**720**	1080	**725**	1090
×1	30.0	22.5	**647**	972	**653**	979
×$^7/_8$	26.4	19.8	**569**	855	**574**	861
×$^3/_4$	22.8	17.1	**491**	739	**496**	744
×$^5/_8$	19.2	14.4	**414**	622	**418**	626
×$^9/_{16}$	17.4	13.0	**375**	564	**377**	566
×$^1/_2$	15.5	11.6	**334**	502	**336**	505
2L8×6×1	26.0	19.5	**560**	842	**566**	848
×$^7/_8$	23.0	17.3	**496**	745	**502**	753
×$^3/_4$	19.9	14.9	**429**	645	**432**	648
×$^5/_8$	16.7	12.5	**360**	541	**363**	544
×$^9/_{16}$	15.1	11.3	**326**	489	**328**	492
×$^1/_2$	13.5	10.1	**291**	437	**293**	439
×$^7/_{16}$	11.9	8.92	**257**	386	**259**	388
2L8×4×1	22.0	16.5	**474**	713	**479**	718
×$^7/_8$	19.5	14.6	**420**	632	**423**	635
×$^3/_4$	16.9	12.7	**364**	548	**368**	552
×$^5/_8$	14.2	10.6	**306**	460	**307**	461
×$^9/_{16}$	12.9	9.68	**278**	418	**281**	421
×$^1/_2$	11.5	8.63	**248**	373	**250**	375
×$^7/_{16}$	10.1	7.58	**218**	327	**220**	330
2L7×4×$^3/_4$	15.4	11.6	**332**	499	**336**	505
×$^5/_8$	13.0	9.75	**280**	421	**283**	424
×$^1/_2$	10.5	7.88	**226**	340	**229**	343
×$^7/_{16}$	9.24	6.93	**199**	299	**201**	301
×$^3/_8$	7.96	5.97	**172**	258	**173**	260
2L6×6×1	22.0	16.5	**474**	713	**479**	718
×$^7/_8$	19.5	14.6	**420**	632	**423**	635
×$^3/_4$	16.9	12.7	**364**	548	**368**	552
×$^5/_8$	14.3	10.7	**308**	463	**310**	465
×$^9/_{16}$	12.9	9.68	**278**	418	**281**	421
×$^1/_2$	11.5	8.63	**248**	373	**250**	375
×$^7/_{16}$	10.2	7.65	**220**	330	**222**	333
×$^3/_8$	8.76	6.57	**189**	284	**191**	286
×$^5/_{16}$	7.34	5.51	**158**	238	**160**	240

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 36 ksi
F_u = 58 ksi

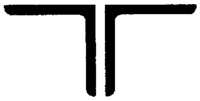
Table 5–8 (continued)
Available Strength in Axial Tension
Double Angles

2L6–2L5

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
2L6×4×7/8	16.0	12.0	**345**	518	**348**	522
×3/4	13.9	10.4	**300**	450	**302**	452
×5/8	11.7	8.78	**252**	379	**255**	382
×9/16	10.6	7.95	**229**	343	**231**	346
×1/2	9.50	7.13	**205**	308	**207**	310
×7/16	8.36	6.27	**180**	271	**182**	273
×3/8	7.22	5.42	**156**	234	**157**	236
×5/16	6.05	4.54	**130**	196	**132**	197
2L6×3 1/2×1/2	9.00	6.75	**194**	292	**196**	294
×3/8	6.84	5.13	**147**	222	**149**	223
×5/16	5.74	4.31	**124**	186	**125**	187
2L5×5×7/8	16.0	12.0	**345**	518	**348**	522
×3/4	13.9	10.4	**300**	450	**302**	452
×5/8	11.7	8.78	**252**	379	**255**	382
×1/2	9.50	7.13	**205**	308	**207**	310
×7/16	8.36	6.27	**180**	271	**182**	273
×3/8	7.22	5.42	**156**	234	**157**	236
×5/16	6.06	4.55	**131**	196	**132**	198
2L5×3 1/2×3/4	11.6	8.70	**250**	376	**252**	378
×5/8	9.84	7.38	**212**	319	**214**	321
×1/2	8.01	6.01	**173**	260	**174**	261
×3/8	6.10	4.57	**131**	198	**133**	199
×5/16	5.12	3.84	**110**	166	**111**	167
×1/4	4.12	3.09	**88.8**	133	**89.6**	134
2L5×3×1/2	7.51	5.63	**162**	243	**163**	245
×7/16	6.62	4.97	**143**	214	**144**	216
×3/8	5.73	4.30	**124**	186	**125**	187
×5/16	4.80	3.60	**103**	156	**104**	157
×1/4	3.88	2.91	**83.6**	126	**84.4**	127

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.



2L4–2L3½

Table 5–8 (continued)
Available Strength in Axial Tension
Double Angles

F_y = 36 ksi
F_u = 58 ksi

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding		Rupture	
			kips		kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
2L4×4×3/4	10.9	8.18	**235**	353	**237**	356
×5/8	9.21	6.91	**199**	298	**200**	301
×1/2	7.49	5.62	**161**	243	**163**	244
×7/16	6.62	4.97	**143**	214	**144**	216
×3/8	5.72	4.29	**123**	185	**124**	187
×5/16	4.80	3.60	**103**	156	**104**	157
×1/4	3.88	2.91	**83.6**	126	**84.4**	127
2L4×3½×1/2	7.01	5.26	**151**	227	**153**	229
×3/8	5.34	4.00	**115**	173	**116**	174
×5/16	4.50	3.38	**97.0**	146	**98.0**	147
×1/4	3.62	2.71	**78.0**	117	**78.6**	118
2L4×3×5/8	7.78	5.84	**168**	252	**169**	254
×1/2	6.50	4.88	**140**	211	**142**	212
×3/8	4.96	3.72	**107**	161	**108**	162
×5/16	4.18	3.14	**90.1**	135	**91.1**	137
×1/4	3.38	2.54	**72.9**	110	**73.7**	110
2L3½×3½×1/2	6.50	4.88	**140**	211	**142**	212
×7/16	5.74	4.31	**124**	186	**125**	187
×3/8	4.96	3.72	**107**	161	**108**	162
×5/16	4.18	3.14	**90.1**	135	**91.1**	137
×1/4	3.38	2.54	**72.9**	110	**73.7**	110
2L3½×3×1/2	6.00	4.50	**129**	194	**131**	196
×7/16	5.30	3.98	**114**	172	**115**	173
×3/8	4.60	3.45	**99.2**	149	**100**	150
×5/16	3.86	2.90	**83.2**	125	**84.1**	126
×1/4	3.12	2.34	**67.3**	101	**67.9**	102
2L3½×2½×1/2	5.50	4.13	**119**	178	**120**	180
×3/8	4.22	3.17	**91.0**	137	**91.9**	138
×5/16	3.56	2.67	**76.7**	115	**77.4**	116
×1/4	2.88	2.16	**62.1**	93.3	**62.6**	94.0

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

F_y = 36 ksi
F_u = 58 ksi

Table 5–8 (continued)
Available Strength in Axial Tension
Double Angles

2L3–2L2

Shape	Gross Area, A_g	$A_e = 0.75A_g$	Yielding kips		Rupture kips	
			P_n/Ω_t	$\phi_t P_n$	P_n/Ω_t	$\phi_t P_n$
	in.^{2}	in.^{2}	ASD	LRFD	ASD	LRFD
2L3×3×1/2	5.50	4.13	119	178	120	180
×7/16	4.86	3.64	105	157	106	158
×3/8	4.22	3.17	91.0	137	91.9	138
×5/16	3.55	2.66	76.5	115	77.1	116
×1/4	2.87	2.15	61.9	93.0	62.4	93.5
×3/16	2.18	1.64	47.0	70.6	47.6	71.3
2L3×2 1/2×1/2	5.00	3.75	108	162	109	163
×7/16	4.42	3.32	95.3	143	96.3	144
×3/8	3.84	2.88	82.8	124	83.5	125
×5/16	3.34	2.51	72.0	108	72.8	109
×1/4	2.62	1.97	56.5	84.9	57.1	85.7
×3/16	1.99	1.49	42.9	64.5	43.2	64.8
2L3×2×1/2	4.50	3.38	97.0	146	98.0	147
×3/8	3.46	2.59	74.6	112	75.1	113
×5/16	2.92	2.19	62.9	94.6	63.5	95.3
×1/4	2.38	1.79	51.3	77.1	51.9	77.9
×3/16	1.80	1.35	38.8	58.3	39.2	58.7
2L2 1/2×2 1/2×1/2	4.50	3.38	97.0	146	98.0	147
×3/8	3.47	2.60	74.8	112	75.4	113
×5/16	2.93	2.20	63.2	94.9	63.8	95.7
×1/4	2.37	1.78	51.1	76.8	51.6	77.4
×3/16	1.80	1.35	38.8	58.3	39.2	58.7
2L2 1/2×2×3/8	3.10	2.33	66.8	100	67.6	101
×5/16	2.62	1.97	56.5	84.9	57.1	85.7
×1/4	2.12	1.59	45.7	68.7	46.1	69.2
×3/16	1.62	1.22	34.9	52.5	35.4	53.1
2L2 1/2×1 1/2×1/4	1.88	1.41	40.5	60.9	40.9	61.3
×3/16	1.43	1.07	30.8	46.3	31.0	46.5
2L2×2×3/8	2.72	2.04	58.6	88.1	59.2	88.7
×5/16	2.30	1.73	49.6	74.5	50.2	75.3
×1/4	1.88	1.41	40.5	60.9	40.9	61.3
×3/16	1.43	1.07	30.8	46.3	31.0	46.5
×1/8	0.968	0.726	20.9	31.4	21.1	31.6

Limit State	ASD	LRFD
Yielding	$\Omega_t = 1.67$	$\phi_t = 0.90$
Rupture	$\Omega_t = 2.00$	$\phi_t = 0.75$

Note: Tensile rupture on the effective net area will control over tensile yielding on the gross area unless the tension member is selected so that an end connection can be configured with $A_e \geq 0.745A_g$.

PART 6

DESIGN OF MEMBERS SUBJECT TO COMBINED LOADING

SCOPE 6–2

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of members subject to combined loading. For the design of members subject to axial tension only, see Part 5. For the design of members subject to axial compression only, see Part 4. For the design of members subject to uniaxial flexure only, see Part 3. For members that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

COMPACT, NON-COMPACT, AND SLENDER-ELEMENT CROSS-SECTIONS

Based upon the types of load transmitted by the member, the discussions of width-thickness ratios in Part 4 for compression members and Part 3 for flexural members apply to the design of members subject to combined loading.

MEMBERS SUBJECT TO COMBINED AXIAL COMPRESSION AND FLEXURE

The interaction of the combined effects of the required strengths (axial compression and bending moment) must satisfy the unity check as follows:

1. For doubly symmetric and singly symmetric members, per AISC Specification Section H1.1.
2. For unsymmetric and other members, per AISC Specification Section H2.

MEMBERS SUBJECT TO COMBINED AXIAL TENSION AND FLEXURE

The interaction of the combined effects of the required strengths (axial tension and bending moment) must satisfy the unity check as follows:

1. For doubly symmetric and singly symmetric members, per AISC Specification Section H1.2.
2. For unsymmetric and other members, per AISC Specification Section H2.

MEMBERS SUBJECT TO COMBINED TORSION, FLEXURE, SHEAR, AND/OR AXIAL FORCE

The interaction of the combined effects of the required strengths (torsion, bending moment, shear force, and/or axial force) must satisfy the requirements of AISC Specification Section H3.

See also AISC Design Guide No. 9 *Torsional Analysis of Structural Steel Members*.

COMPOSITE MEMBERS SUBJECT TO COMBINED AXIAL COMPRESSION AND FLEXURE

For the design of composite members subject to combined axial compression and flexure, see AISC Specification Section I4.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

Based upon the types of load transmitted by the member, the specification requirements and design considerations given in Part 5 for tension members, Part 4 for compression members, and Part 3 for flexural members apply to the design of members subject to combined loading.

STEEL BEAM-COLUMN SELECTION TABLES

Table 6–1. W-Shapes in Combined Axial and Bending

The determination of the adequacy of W-shapes subject to combined axial and bending loads is facilitated by the use of this table. The AISC Specification Equations to check the adequacy of W-shapes with $F_y = 50$ ksi (ASTM A992) subject to combined axial force and flexure can be determined using the values of p, b_x, b_y, t_r and t_y, tabulated in Table 6-1. These variables are defined as follows:

	LRFD	ASD
Axial Compression	$p = \dfrac{1}{\phi_c P_n}$, (kips)$^{-1}$	$p = \dfrac{\Omega_c}{P_n}$, (kips)$^{-1}$
Strong Axis Bending	$b_x = \dfrac{8}{9\phi_b M_{nx}}$, (kip-ft)$^{-1}$	$b_x = \dfrac{8\Omega_b}{9M_{nx}}$, (kip-ft)$^{-1}$
Weak Axis Bending	$b_y = \dfrac{8}{9\phi_b M_{ny}}$, (kip-ft)$^{-1}$	$b_y = \dfrac{8\Omega_b}{9M_{ny}}$, (kip-ft)$^{-1}$
Tension Rupture	$t_r = \dfrac{1}{\phi_t 0.75 F_u A_g}$, (kips)$^{-1}$	$t_r = \dfrac{\Omega_t}{0.75 F_u A_g}$, (kips)$^{-1}$
Tension Yielding	$t_y = \dfrac{1}{\phi_c F_y A_g}$, (kips)$^{-1}$	$t_y = \dfrac{\Omega}{F_y A_g}$, (kips)$^{-1}$

Table 6-1 is normally used with iteration to determine an appropriate shape. After selecting a trial shape, the sum of the load ratios reveals if that trial shape is close (a sum nearly equal to 1.0), conservative (a sum less than 1.0), or unconservative (a sum greater than 1.0). When the trial shape is unconservative and axial effects dominate, the second trial shape should be one with a larger value of p. Similarly, when X-X axis or Y-Y axis flexural effects dominate, the second trial shape should be one with a larger value of b_x or b_y, respectively. This process can be repeated until an acceptable shape is determined.

An alternative approach for the initial selection of members may be found in Aminmansour (2000).

Combined Compression and Flexure

In this case, the compressive component of the combined force is accounted for by selecting the proper value of p based on the larger of the effective length for compression buckling about the Y-Y axis $(KL)_y$ and the effective length for compression buckling about the X-X axis $(KL)_{y\,eq}$, as described in Part 4. The tabulated values can be used directly, as the slenderness of the cross section is accounted for.

The bending component of the combined force is accounted for by selecting the proper value of b_x based on the unbraced length L_b, as described in Part 3. Because unbraced length is not a factor in weak-axis bending, a single value of b_y applies for any given W-shape.

When $P_r/P_c \geq 0.2$, the tabulated values of p, b_x, and b_y can be used as follows to solve the modified form of AISC Specification Equation H1-1a:

$$pP_r + b_xM_{rx} + b_yM_{ry} \leq 1.0$$

When $P_r/P_c < 0.2$, the tabulated values of p, b_x, and b_y can be used as follows to solve the modified form of AISC Specification Equation H1-1b:

$$\tfrac{1}{2}pP_r + \tfrac{9}{8}(b_xM_{rx} + b_yM_{ry}) \leq 1.0$$

The tabulated values of b_x and b_y assume that $C_b = 1.0$. These values may be modified in accordance with AISC Specification Section F1.

For further information, see Aminmansour (2000).

Combined Tension and Flexure

In this case, the axial component of the combined force is accounted for by selecting the larger value of t_y *or* t_r, for the critical case of available tension yield or available tension rupture strength, as described in Part 5. It is important to note that the tabulated values for t_r are based upon the assumption that $A_e = 0.75A_g$, which is arbitrarily selected as a value that is practical to achieve with typical end connections. When $A_e > 0.75A_g$, the tabulated values for t_r can be used conservatively. When $A_e < 0.75A_g$, the tabulated values of t_r cannot be used, but rather, must be calculated based upon the actual value of A_e.

The bending component of the combined force is accounted for by selecting the proper value of b_x for the unbraced length L_b, as described in Part 3. Because unbraced length is not a factor in weak-axis bending, a single value of b_y applies for any given W-shape.

When $P_r/P_c \geq 0.2$, the tabulated values of t_y, t_r, b_x, and b_y can be used as follows to solve the modified form of AISC Specification Equation H1-1a:

$$(t_y \text{ or } t_r)\, P_r + b_xM_{rx} + b_yM_{ry} \leq 1.0$$

When $P_r/P_c < 0.2$, the tabulated values of p, b_x, and b_y can be used as follows to solve the modified form of AISC Specification Equation H1-1b:

$$\tfrac{1}{2}(t_y \text{ or } t_r)\, P_r + \tfrac{9}{8}(b_xM_{rx} + b_yM_{ry}) \leq 1.0$$

The tabulated values of b_x and b_y assume that $C_b = 1.0$. These values may be modified in accordance with AISC Specification Sections F1 and H1.2.

PART 6 REFERENCES

Aminmansour, A., 2000, "A New Approach for Design of Steel Beam-Columns," *Engineering Journal*, Vol. 37, No. 2, (2nd Qtr.), pp. 41-72, AISC, Chicago, IL.

F_y = 50 ksi

Table 6–1
Combined Axial and Bending
W Shapes

W44

Shape		W44×											
		335^c				290^c				262^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.345**	0.230	**0.220**	0.146	**0.417**	0.278	**0.253**	0.168	**0.476**	0.317	**0.281**	0.187
	11	**0.378**	0.251	**0.220**	0.146	**0.454**	0.302	**0.253**	0.168	**0.518**	0.344	**0.281**	0.187
	12	**0.384**	0.256	**0.220**	0.146	**0.462**	0.307	**0.253**	0.168	**0.526**	0.350	**0.281**	0.187
	13	**0.393**	0.261	**0.222**	0.148	**0.470**	0.313	**0.255**	0.170	**0.536**	0.356	**0.284**	0.189
	14	**0.402**	0.267	**0.225**	0.150	**0.480**	0.319	**0.259**	0.173	**0.546**	0.363	**0.289**	0.192
	15	**0.412**	0.274	**0.229**	0.152	**0.490**	0.326	**0.264**	0.175	**0.557**	0.371	**0.294**	0.196
	16	**0.423**	0.282	**0.233**	0.155	**0.501**	0.333	**0.268**	0.178	**0.570**	0.379	**0.299**	0.199
	17	**0.435**	0.290	**0.236**	0.157	**0.514**	0.342	**0.273**	0.181	**0.584**	0.389	**0.304**	0.203
	18	**0.449**	0.299	**0.240**	0.160	**0.527**	0.351	**0.277**	0.184	**0.599**	0.399	**0.310**	0.206
	19	**0.463**	0.308	**0.244**	0.162	**0.542**	0.361	**0.282**	0.188	**0.616**	0.410	**0.316**	0.210
	20	**0.479**	0.319	**0.248**	0.165	**0.559**	0.372	**0.287**	0.191	**0.634**	0.422	**0.322**	0.214
	22	**0.515**	0.343	**0.257**	0.171	**0.597**	0.397	**0.298**	0.198	**0.676**	0.450	**0.335**	0.223
	24	**0.558**	0.371	**0.266**	0.177	**0.644**	0.428	**0.309**	0.206	**0.727**	0.484	**0.348**	0.232
	26	**0.608**	0.405	**0.275**	0.183	**0.702**	0.467	**0.321**	0.214	**0.788**	0.524	**0.363**	0.242
	28	**0.668**	0.444	**0.286**	0.190	**0.770**	0.513	**0.334**	0.223	**0.862**	0.574	**0.380**	0.253
	30	**0.738**	0.491	**0.297**	0.198	**0.852**	0.567	**0.349**	0.232	**0.954**	0.635	**0.397**	0.264
	32	**0.822**	0.547	**0.310**	0.206	**0.948**	0.631	**0.365**	0.243	**1.06**	0.708	**0.417**	0.278
	34	**0.923**	0.614	**0.323**	0.215	**1.06**	0.708	**0.382**	0.254	**1.20**	0.796	**0.439**	0.292
	36	**1.04**	0.689	**0.338**	0.225	**1.19**	0.794	**0.401**	0.267	**1.34**	0.892	**0.466**	0.310
	38	**1.15**	0.767	**0.354**	0.235	**1.33**	0.885	**0.429**	0.285	**1.49**	0.994	**0.508**	0.338
	40	**1.28**	0.850	**0.377**	0.251	**1.47**	0.981	**0.463**	0.308	**1.66**	1.10	**0.550**	0.366
	42	**1.41**	0.937	**0.405**	0.269	**1.62**	1.08	**0.498**	0.331	**1.83**	1.21	**0.593**	0.394
	44	**1.55**	1.03	**0.432**	0.287	**1.78**	1.19	**0.533**	0.355	**2.00**	1.33	**0.636**	0.423
	46	**1.69**	1.12	**0.459**	0.306	**1.95**	1.30	**0.569**	0.378	**2.19**	1.46	**0.679**	0.452
	48	**1.84**	1.22	**0.487**	0.324	**2.12**	1.41	**0.604**	0.402	**2.38**	1.59	**0.723**	0.481
	50	**2.00**	1.33	**0.514**	0.342	**2.30**	1.53	**0.640**	0.426	**2.59**	1.72	**0.767**	0.510

Other Constants and Properties

	335^c ASD	335^c LRFD	290^c ASD	290^c LRFD	262^c ASD	262^c LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	**1.51**	1.00	**1.74**	1.16	**1.96**	1.30
$t_y \times 10^3$ (kips)$^{-1}$	**0.339**	0.226	**0.390**	0.260	**0.434**	0.289
$t_r \times 10^3$ (kips)$^{-1}$	**0.417**	0.278	**0.480**	0.320	**0.534**	0.356
r_x/r_y	5.10		5.10		5.10	

^{c} Shape is slender for compression with F_y = 50 ksi.

W44–W40

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

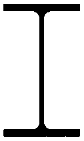
Shape		W44×				W40×							
		230^c				593^h				503^h			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.558	0.371	0.324	0.215	0.192	0.128	0.129	0.0859	0.226	0.150	0.154	0.103
	11	0.605	0.403	0.324	0.215	0.210	0.139	0.129	0.0859	0.247	0.165	0.154	0.103
	12	0.615	0.409	0.324	0.215	0.213	0.142	0.129	0.0859	0.252	0.168	0.154	0.103
	13	0.626	0.417	0.329	0.219	0.217	0.144	0.129	0.0859	0.257	0.171	0.154	0.103
	14	0.638	0.424	0.335	0.223	0.221	0.147	0.130	0.0863	0.262	0.174	0.155	0.103
	15	0.651	0.433	0.341	0.227	0.226	0.150	0.131	0.0870	0.268	0.178	0.157	0.104
	16	0.666	0.443	0.347	0.231	0.231	0.154	0.132	0.0877	0.274	0.182	0.159	0.105
	17	0.682	0.454	0.354	0.235	0.237	0.158	0.133	0.0884	0.281	0.187	0.160	0.107
	18	0.700	0.465	0.360	0.240	0.243	0.162	0.134	0.0892	0.289	0.192	0.162	0.108
	19	0.719	0.478	0.367	0.244	0.250	0.166	0.135	0.0899	0.297	0.198	0.163	0.109
	20	0.740	0.492	0.375	0.249	0.257	0.171	0.136	0.0907	0.306	0.204	0.165	0.110
	22	0.789	0.525	0.390	0.260	0.273	0.182	0.139	0.0923	0.326	0.217	0.168	0.112
	24	0.847	0.563	0.407	0.271	0.292	0.194	0.141	0.0939	0.350	0.233	0.172	0.114
	26	0.917	0.610	0.425	0.283	0.314	0.209	0.144	0.0956	0.377	0.251	0.176	0.117
	28	1.00	0.667	0.445	0.296	0.340	0.226	0.146	0.0973	0.410	0.273	0.180	0.119
	30	1.11	0.736	0.468	0.311	0.370	0.246	0.149	0.0991	0.448	0.298	0.184	0.122
	32	1.23	0.821	0.492	0.327	0.405	0.269	0.152	0.101	0.492	0.327	0.188	0.125
	34	1.39	0.925	0.519	0.345	0.446	0.297	0.155	0.103	0.544	0.362	0.192	0.128
	36	1.56	1.04	0.567	0.377	0.494	0.329	0.158	0.105	0.606	0.403	0.197	0.131
	38	1.74	1.16	0.620	0.413	0.551	0.366	0.161	0.107	0.675	0.449	0.202	0.134
	40	1.92	1.28	0.674	0.448	0.610	0.406	0.164	0.109	0.748	0.498	0.207	0.138
	42	2.12	1.41	0.728	0.485	0.673	0.448	0.168	0.112	0.825	0.549	0.212	0.141
	44	2.33	1.55	0.784	0.521	0.738	0.491	0.171	0.114	0.906	0.603	0.218	0.145
	46	2.55	1.69	0.839	0.558	0.807	0.537	0.175	0.116	0.990	0.659	0.224	0.149
	48	2.77	1.84	0.896	0.596	0.879	0.585	0.179	0.119	1.08	0.717	0.231	0.153
	50	3.01	2.00	0.953	0.634	0.953	0.634	0.183	0.122	1.17	0.778	0.237	0.158
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		2.27		1.51		0.741		0.493		0.904		0.602	
$t_y \times 10^3$ (kips)$^{-1}$		0.492		0.328		0.192		0.128		0.225		0.150	
$t_r \times 10^3$ (kips)$^{-1}$		0.606		0.404		0.236		0.157		0.278		0.185	
r_x/r_y		5.10				4.47				4.52			

[c] Shape is slender for compression with F_y = 50 ksi.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6-1 (continued)
Combined Axial and Bending
W Shapes


W40

Shape		W40×											
		431^h				397^h				392^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.263	0.175	0.182	0.121	0.285	0.190	0.198	0.132	0.290	0.193	0.208	0.139
	11	0.289	0.193	0.182	0.121	0.314	0.209	0.198	0.132	0.349	0.232	0.213	0.142
	12	0.295	0.196	0.182	0.121	0.320	0.213	0.198	0.132	0.361	0.240	0.217	0.144
	13	0.301	0.200	0.182	0.121	0.327	0.217	0.198	0.132	0.375	0.249	0.220	0.146
	14	0.307	0.204	0.184	0.122	0.334	0.222	0.201	0.133	0.391	0.260	0.223	0.148
	15	0.314	0.209	0.186	0.124	0.341	0.227	0.203	0.135	0.408	0.271	0.227	0.151
	16	0.322	0.214	0.188	0.125	0.350	0.233	0.205	0.137	0.428	0.284	0.230	0.153
	17	0.330	0.220	0.190	0.127	0.359	0.239	0.208	0.138	0.449	0.299	0.234	0.156
	18	0.340	0.226	0.193	0.128	0.369	0.246	0.211	0.140	0.474	0.315	0.238	0.158
	19	0.350	0.233	0.195	0.130	0.380	0.253	0.213	0.142	0.501	0.333	0.241	0.161
	20	0.361	0.240	0.197	0.131	0.392	0.261	0.216	0.144	0.531	0.354	0.245	0.163
	22	0.386	0.257	0.202	0.134	0.419	0.279	0.222	0.147	0.603	0.401	0.254	0.169
	24	0.415	0.276	0.207	0.138	0.451	0.300	0.227	0.151	0.693	0.461	0.263	0.175
	26	0.449	0.299	0.212	0.141	0.488	0.325	0.234	0.156	0.808	0.538	0.273	0.181
	28	0.489	0.325	0.218	0.145	0.532	0.354	0.240	0.160	0.937	0.623	0.283	0.188
	30	0.536	0.356	0.224	0.149	0.584	0.388	0.247	0.165	1.08	0.716	0.295	0.196
	32	0.591	0.393	0.230	0.153	0.644	0.429	0.255	0.169	1.22	0.814	0.307	0.204
	34	0.656	0.436	0.237	0.157	0.715	0.476	0.263	0.175	1.38	0.919	0.320	0.213
	36	0.734	0.488	0.243	0.162	0.801	0.533	0.271	0.180	1.55	1.03	0.335	0.223
	38	0.818	0.544	0.251	0.167	0.892	0.594	0.280	0.186	1.73	1.15	0.351	0.233
	40	0.906	0.603	0.259	0.172	0.989	0.658	0.289	0.193	1.91	1.27	0.372	0.247
	42	0.999	0.665	0.267	0.178	1.09	0.725	0.300	0.199	2.11	1.40	0.393	0.262
	44	1.10	0.729	0.276	0.184	1.20	0.796	0.311	0.207	2.31	1.54	0.415	0.276
	46	1.20	0.797	0.285	0.190	1.31	0.870	0.322	0.215				
	48	1.30	0.868	0.296	0.197	1.42	0.947	0.339	0.225				
	50	1.42	0.942	0.309	0.205	1.55	1.03	0.356	0.237				

Other Constants and Properties

	431^h		397^h		392^h	
$b_y \times 10^3$ (kip-ft)$^{-1}$	1.09	0.723	1.19	0.790	1.71	1.14
$t_y \times 10^3$ (kips)$^{-1}$	0.263	0.175	0.285	0.190	0.290	0.193
$t_r \times 10^3$ (kips)$^{-1}$	0.323	0.215	0.351	0.234	0.357	0.238
r_x/r_y	4.55		4.56		6.10	

^h Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W40

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W40×											
		372^h				362^h				331^h			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**0.306**	0.204	**0.212**	0.141	**0.312**	0.208	**0.217**	0.145	**0.343**	0.228	**0.249**	0.166
	11	**0.338**	0.225	**0.212**	0.141	**0.344**	0.229	**0.217**	0.145	**0.416**	0.276	**0.257**	0.171
	12	**0.344**	0.229	**0.212**	0.141	**0.351**	0.233	**0.217**	0.145	**0.431**	0.287	**0.262**	0.174
	13	**0.352**	0.234	**0.213**	0.142	**0.358**	0.238	**0.218**	0.145	**0.449**	0.298	**0.266**	0.177
	14	**0.359**	0.239	**0.215**	0.143	**0.366**	0.244	**0.221**	0.147	**0.468**	0.312	**0.271**	0.180
	15	**0.368**	0.245	**0.218**	0.145	**0.375**	0.249	**0.224**	0.149	**0.490**	0.326	**0.276**	0.184
	16	**0.377**	0.251	**0.221**	0.147	**0.384**	0.256	**0.227**	0.151	**0.515**	0.343	**0.281**	0.187
	17	**0.388**	0.258	**0.224**	0.149	**0.395**	0.263	**0.230**	0.153	**0.543**	0.361	**0.287**	0.191
	18	**0.399**	0.265	**0.227**	0.151	**0.406**	0.270	**0.233**	0.155	**0.574**	0.382	**0.292**	0.194
	19	**0.411**	0.273	**0.230**	0.153	**0.419**	0.278	**0.236**	0.157	**0.609**	0.405	**0.298**	0.198
	20	**0.424**	0.282	**0.233**	0.155	**0.432**	0.287	**0.239**	0.159	**0.648**	0.431	**0.304**	0.202
	22	**0.454**	0.302	**0.239**	0.159	**0.463**	0.308	**0.246**	0.164	**0.741**	0.493	**0.317**	0.211
	24	**0.489**	0.326	**0.246**	0.164	**0.498**	0.332	**0.253**	0.168	**0.858**	0.571	**0.331**	0.220
	26	**0.531**	0.353	**0.254**	0.169	**0.541**	0.360	**0.261**	0.174	**1.01**	0.669	**0.346**	0.230
	28	**0.579**	0.385	**0.261**	0.174	**0.590**	0.393	**0.269**	0.179	**1.17**	0.776	**0.363**	0.241
	30	**0.637**	0.424	**0.269**	0.179	**0.648**	0.431	**0.278**	0.185	**1.34**	0.891	**0.381**	0.253
	32	**0.704**	0.468	**0.278**	0.185	**0.717**	0.477	**0.287**	0.191	**1.52**	1.01	**0.401**	0.267
	34	**0.784**	0.521	**0.287**	0.191	**0.798**	0.531	**0.297**	0.197	**1.72**	1.14	**0.425**	0.283
	36	**0.879**	0.585	**0.297**	0.198	**0.895**	0.596	**0.307**	0.204	**1.93**	1.28	**0.457**	0.304
	38	**0.979**	0.652	**0.308**	0.205	**0.998**	0.664	**0.319**	0.212	**2.15**	1.43	**0.488**	0.325
	40	**1.09**	0.722	**0.319**	0.213	**1.11**	0.735	**0.331**	0.220	**2.38**	1.58	**0.519**	0.346
	42	**1.20**	0.796	**0.332**	0.221	**1.22**	0.811	**0.344**	0.229	**2.62**	1.75	**0.551**	0.366
	44	**1.31**	0.874	**0.345**	0.230	**1.34**	0.890	**0.359**	0.239				
	46	**1.44**	0.955	**0.364**	0.242	**1.46**	0.973	**0.380**	0.253				
	48	**1.56**	1.04	**0.384**	0.256	**1.59**	1.06	**0.401**	0.267				
	50	**1.70**	1.13	**0.404**	0.269	**1.73**	1.15	**0.422**	0.281				

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	**1.29**	0.856	**1.32**	0.878	**2.10**	1.40
$t_y \times 10^3$ (kips)$^{-1}$	**0.306**	0.204	**0.312**	0.208	**0.342**	0.228
$t_r \times 10^3$ (kips)$^{-1}$	**0.377**	0.251	**0.384**	0.256	**0.422**	0.281
r_x/r_y	4.58		4.58		6.19	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W40

Shape		W40×											
		327^h				324				297^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**0.348**	0.232	**0.253**	0.168	**0.350**	0.233	**0.244**	0.162	**0.385**	0.256	**0.268**	0.178
	11	**0.421**	0.280	**0.261**	0.174	**0.387**	0.257	**0.244**	0.162	**0.423**	0.281	**0.268**	0.178
	12	**0.437**	0.291	**0.265**	0.177	**0.394**	0.262	**0.244**	0.162	**0.431**	0.287	**0.268**	0.178
	13	**0.455**	0.302	**0.270**	0.180	**0.403**	0.268	**0.245**	0.163	**0.440**	0.293	**0.270**	0.179
	14	**0.474**	0.316	**0.275**	0.183	**0.412**	0.274	**0.249**	0.165	**0.451**	0.300	**0.274**	0.182
	15	**0.497**	0.331	**0.280**	0.186	**0.421**	0.280	**0.252**	0.168	**0.462**	0.307	**0.278**	0.185
	16	**0.522**	0.347	**0.285**	0.190	**0.432**	0.288	**0.256**	0.170	**0.474**	0.315	**0.282**	0.188
	17	**0.550**	0.366	**0.290**	0.193	**0.444**	0.296	**0.259**	0.172	**0.487**	0.324	**0.286**	0.190
	18	**0.581**	0.387	**0.296**	0.197	**0.457**	0.304	**0.263**	0.175	**0.502**	0.334	**0.291**	0.193
	19	**0.616**	0.410	**0.302**	0.201	**0.471**	0.314	**0.267**	0.178	**0.518**	0.344	**0.295**	0.196
	20	**0.655**	0.436	**0.308**	0.205	**0.487**	0.324	**0.271**	0.180	**0.535**	0.356	**0.300**	0.200
	22	**0.748**	0.498	**0.321**	0.213	**0.521**	0.347	**0.279**	0.186	**0.574**	0.382	**0.310**	0.206
	24	**0.865**	0.576	**0.335**	0.223	**0.562**	0.374	**0.288**	0.192	**0.620**	0.413	**0.320**	0.213
	26	**1.01**	0.674	**0.350**	0.233	**0.611**	0.406	**0.298**	0.198	**0.674**	0.449	**0.332**	0.221
	28	**1.18**	0.782	**0.367**	0.244	**0.667**	0.444	**0.308**	0.205	**0.738**	0.491	**0.344**	0.229
	30	**1.35**	0.898	**0.385**	0.256	**0.734**	0.488	**0.318**	0.212	**0.814**	0.542	**0.357**	0.237
	32	**1.54**	1.02	**0.406**	0.270	**0.813**	0.541	**0.330**	0.220	**0.903**	0.601	**0.371**	0.247
	34	**1.73**	1.15	**0.430**	0.286	**0.906**	0.603	**0.343**	0.228	**1.01**	0.673	**0.386**	0.257
	36	**1.94**	1.29	**0.462**	0.308	**1.02**	0.676	**0.356**	0.237	**1.13**	0.754	**0.403**	0.268
	38	**2.17**	1.44	**0.494**	0.329	**1.13**	0.753	**0.371**	0.247	**1.26**	0.840	**0.421**	0.280
	40	**2.40**	1.60	**0.526**	0.350	**1.25**	0.835	**0.386**	0.257	**1.40**	0.931	**0.444**	0.296
	42	**2.65**	1.76	**0.558**	0.371	**1.38**	0.920	**0.407**	0.271	**1.54**	1.03	**0.476**	0.317
	44					**1.52**	1.01	**0.433**	0.288	**1.69**	1.13	**0.508**	0.338
	46					**1.66**	1.10	**0.460**	0.306	**1.85**	1.23	**0.539**	0.359
	48					**1.81**	1.20	**0.486**	0.324	**2.02**	1.34	**0.571**	0.380
	50					**1.96**	1.30	**0.513**	0.341	**2.19**	1.45	**0.603**	0.401

Other Constants and Properties

	327^h		324		297^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**2.12**	1.41	**1.49**	0.992	**1.66**	1.10
$t_y \times 10^3$ (kips)$^{-1}$	**0.348**	0.232	**0.350**	0.233	**0.381**	0.254
$t_r \times 10^3$ (kips)$^{-1}$	**0.428**	0.285	**0.431**	0.287	**0.470**	0.313
r_x/r_y	6.20		4.58		4.60	

[c] Shape is slender for compression with F_y = 50 ksi.

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

W40

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W40×											
		294				278				277^c			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.387**	0.258	**0.281**	0.187	**0.407**	0.271	**0.299**	0.199	**0.425**	0.283	**0.285**	0.190
	11	**0.471**	0.313	**0.291**	0.194	**0.498**	0.331	**0.312**	0.207	**0.463**	0.308	**0.285**	0.190
	12	**0.489**	0.325	**0.296**	0.197	**0.517**	0.344	**0.318**	0.211	**0.471**	0.313	**0.285**	0.190
	13	**0.509**	0.339	**0.302**	0.201	**0.539**	0.359	**0.324**	0.216	**0.479**	0.319	**0.287**	0.191
	14	**0.532**	0.354	**0.308**	0.205	**0.564**	0.375	**0.331**	0.220	**0.489**	0.325	**0.291**	0.193
	15	**0.557**	0.371	**0.314**	0.209	**0.592**	0.394	**0.338**	0.225	**0.499**	0.332	**0.295**	0.196
	16	**0.586**	0.390	**0.321**	0.214	**0.623**	0.414	**0.345**	0.229	**0.510**	0.340	**0.300**	0.199
	17	**0.618**	0.411	**0.328**	0.218	**0.658**	0.438	**0.352**	0.234	**0.523**	0.348	**0.305**	0.203
	18	**0.654**	0.435	**0.335**	0.223	**0.697**	0.464	**0.360**	0.240	**0.537**	0.357	**0.309**	0.206
	19	**0.695**	0.462	**0.342**	0.228	**0.741**	0.493	**0.369**	0.245	**0.552**	0.367	**0.314**	0.209
	20	**0.740**	0.492	**0.350**	0.233	**0.791**	0.526	**0.377**	0.251	**0.570**	0.379	**0.320**	0.213
	22	**0.848**	0.564	**0.366**	0.244	**0.909**	0.605	**0.396**	0.263	**0.611**	0.406	**0.330**	0.220
	24	**0.984**	0.655	**0.384**	0.256	**1.06**	0.705	**0.416**	0.277	**0.659**	0.438	**0.342**	0.228
	26	**1.15**	0.768	**0.404**	0.269	**1.24**	0.828	**0.439**	0.292	**0.715**	0.476	**0.355**	0.236
	28	**1.34**	0.891	**0.426**	0.284	**1.44**	0.960	**0.464**	0.309	**0.781**	0.520	**0.368**	0.245
	30	**1.54**	1.02	**0.451**	0.300	**1.66**	1.10	**0.493**	0.328	**0.860**	0.572	**0.383**	0.255
	32	**1.75**	1.16	**0.482**	0.320	**1.88**	1.25	**0.535**	0.356	**0.952**	0.633	**0.398**	0.265
	34	**1.97**	1.31	**0.521**	0.347	**2.13**	1.42	**0.579**	0.386	**1.06**	0.706	**0.415**	0.276
	36	**2.21**	1.47	**0.561**	0.373	**2.38**	1.59	**0.624**	0.415	**1.19**	0.792	**0.434**	0.289
	38	**2.47**	1.64	**0.600**	0.400	**2.66**	1.77	**0.669**	0.445	**1.33**	0.882	**0.454**	0.302
	40	**2.73**	1.82	**0.640**	0.426	**2.94**	1.96	**0.713**	0.475	**1.47**	0.978	**0.485**	0.322
	42	**3.01**	2.00	**0.679**	0.452	**3.25**	2.16	**0.758**	0.504	**1.62**	1.08	**0.520**	0.346
	44									**1.78**	1.18	**0.555**	0.369
	46									**1.94**	1.29	**0.591**	0.393
	48									**2.12**	1.41	**0.626**	0.417
	50									**2.30**	1.53	**0.662**	0.440

Other Constants and Properties

	294		278		277^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**2.38**	1.58	**2.56**	1.70	**1.75**	1.16
$t_y \times 10^3$ (kips)$^{-1}$	**0.387**	0.258	**0.407**	0.271	**0.410**	0.273
$t_r \times 10^3$ (kips)$^{-1}$	**0.476**	0.317	**0.501**	0.334	**0.504**	0.336
r_x/r_y	6.24		6.27		4.58	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W40

Shape		W40×											
		264				249^c				235^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.431**	0.287	**0.315**	0.210	**0.484**	0.322	**0.318**	0.212	**0.505**	0.336	**0.353**	0.235
	11	**0.526**	0.350	**0.329**	0.219	**0.526**	0.350	**0.318**	0.212	**0.596**	0.397	**0.368**	0.245
	12	**0.547**	0.364	**0.335**	0.223	**0.535**	0.356	**0.318**	0.212	**0.616**	0.410	**0.376**	0.250
	13	**0.570**	0.379	**0.342**	0.228	**0.544**	0.362	**0.320**	0.213	**0.639**	0.425	**0.384**	0.256
	14	**0.596**	0.397	**0.349**	0.233	**0.555**	0.369	**0.325**	0.217	**0.667**	0.444	**0.393**	0.261
	15	**0.625**	0.416	**0.357**	0.238	**0.567**	0.377	**0.331**	0.220	**0.699**	0.465	**0.402**	0.267
	16	**0.658**	0.438	**0.365**	0.243	**0.580**	0.386	**0.336**	0.224	**0.735**	0.489	**0.411**	0.274
	17	**0.695**	0.463	**0.373**	0.248	**0.594**	0.395	**0.342**	0.227	**0.776**	0.516	**0.421**	0.280
	18	**0.737**	0.490	**0.382**	0.254	**0.609**	0.405	**0.347**	0.231	**0.822**	0.547	**0.432**	0.287
	19	**0.784**	0.521	**0.391**	0.260	**0.626**	0.417	**0.353**	0.235	**0.873**	0.581	**0.443**	0.294
	20	**0.836**	0.556	**0.401**	0.267	**0.645**	0.429	**0.359**	0.239	**0.930**	0.619	**0.454**	0.302
	22	**0.961**	0.639	**0.421**	0.280	**0.687**	0.457	**0.372**	0.248	**1.07**	0.710	**0.479**	0.319
	24	**1.12**	0.745	**0.444**	0.295	**0.738**	0.491	**0.386**	0.257	**1.24**	0.825	**0.507**	0.337
	26	**1.32**	0.875	**0.469**	0.312	**0.801**	0.533	**0.401**	0.267	**1.46**	0.968	**0.538**	0.358
	28	**1.53**	1.01	**0.498**	0.331	**0.877**	0.583	**0.417**	0.278	**1.69**	1.12	**0.574**	0.382
	30	**1.75**	1.16	**0.533**	0.355	**0.966**	0.643	**0.435**	0.289	**1.94**	1.29	**0.630**	0.419
	32	**1.99**	1.33	**0.583**	0.388	**1.07**	0.713	**0.454**	0.302	**2.20**	1.47	**0.690**	0.459
	34	**2.25**	1.50	**0.632**	0.421	**1.20**	0.797	**0.475**	0.316	**2.49**	1.66	**0.751**	0.500
	36	**2.52**	1.68	**0.682**	0.454	**1.34**	0.894	**0.498**	0.331	**2.79**	1.86	**0.812**	0.540
	38	**2.81**	1.87	**0.731**	0.486	**1.50**	0.996	**0.530**	0.353	**3.11**	2.07	**0.873**	0.581
	40	**3.11**	2.07	**0.780**	0.519	**1.66**	1.10	**0.573**	0.381	**3.44**	2.29	**0.934**	0.621
	42	**3.43**	2.28	**0.830**	0.552	**1.83**	1.22	**0.616**	0.410	**3.80**	2.53	**0.994**	0.662
	44					**2.01**	1.34	**0.659**	0.439				
	46					**2.19**	1.46	**0.703**	0.468				
	48					**2.39**	1.59	**0.746**	0.497				
	50					**2.59**	1.72	**0.790**	0.526				

Other Constants and Properties

	264 ASD	264 LRFD	249^c ASD	249^c LRFD	235^c ASD	235^c LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	**2.70**	1.80	**1.96**	1.30	**3.02**	2.01
$t_y \times 10^3$ (kips)$^{-1}$	**0.431**	0.287	**0.455**	0.303	**0.483**	0.322
$t_r \times 10^3$ (kips)$^{-1}$	**0.530**	0.353	**0.560**	0.373	**0.596**	0.397
r_x/r_y	6.27		4.59		6.26	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W40

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

$F_y = 50$ ksi

Shape		W40×											
		215[c]				211[c]				199[c]			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.579	0.385	0.370	0.246	0.579	0.385	0.393	0.262	0.633	0.421	0.410	0.273
	11	0.629	0.418	0.370	0.246	0.682	0.454	0.412	0.274	0.688	0.458	0.410	0.273
	12	0.639	0.425	0.370	0.246	0.705	0.469	0.422	0.281	0.700	0.466	0.410	0.273
	13	0.650	0.432	0.373	0.248	0.730	0.486	0.432	0.287	0.712	0.474	0.416	0.277
	14	0.662	0.440	0.379	0.252	0.760	0.505	0.442	0.294	0.726	0.483	0.423	0.282
	15	0.675	0.449	0.385	0.256	0.793	0.527	0.453	0.301	0.742	0.493	0.431	0.287
	16	0.690	0.459	0.392	0.261	0.831	0.553	0.464	0.309	0.759	0.505	0.439	0.292
	17	0.707	0.470	0.399	0.265	0.874	0.581	0.476	0.317	0.777	0.517	0.447	0.297
	18	0.724	0.482	0.406	0.270	0.925	0.616	0.489	0.325	0.797	0.531	0.455	0.303
	19	0.744	0.495	0.413	0.275	0.984	0.655	0.503	0.334	0.820	0.545	0.464	0.309
	20	0.765	0.509	0.421	0.280	1.05	0.699	0.517	0.344	0.844	0.562	0.473	0.315
	22	0.814	0.541	0.437	0.291	1.21	0.804	0.548	0.364	0.900	0.599	0.493	0.328
	24	0.872	0.580	0.455	0.303	1.41	0.940	0.583	0.388	0.968	0.644	0.514	0.342
	26	0.941	0.626	0.474	0.315	1.66	1.10	0.622	0.414	1.05	0.698	0.537	0.357
	28	1.02	0.681	0.494	0.329	1.92	1.28	0.679	0.452	1.15	0.763	0.563	0.374
	30	1.12	0.747	0.517	0.344	2.21	1.47	0.753	0.501	1.27	0.842	0.590	0.393
	32	1.25	0.829	0.542	0.360	2.51	1.67	0.827	0.551	1.41	0.940	0.621	0.413
	34	1.39	0.928	0.569	0.378	2.83	1.89	0.902	0.600	1.59	1.06	0.655	0.436
	36	1.56	1.04	0.604	0.402	3.18	2.11	0.978	0.651	1.78	1.19	0.716	0.476
	38	1.74	1.16	0.659	0.438	3.54	2.36	1.05	0.701	1.99	1.32	0.782	0.521
	40	1.93	1.28	0.714	0.475	3.92	2.61	1.13	0.751	2.20	1.46	0.850	0.565
	42	2.13	1.42	0.770	0.512					2.43	1.61	0.918	0.611
	44	2.34	1.55	0.827	0.550					2.66	1.77	0.988	0.657
	46	2.55	1.70	0.884	0.588					2.91	1.94	1.06	0.704
	48	2.78	1.85	0.941	0.626					3.17	2.11	1.13	0.751
	50	3.02	2.01	0.999	0.665					3.44	2.29	1.20	0.798

Other Constants and Properties

	215[c]		211[c]		199[c]	
$b_y \times 10^3$ (kip-ft)$^{-1}$	2.28	1.52	3.39	2.26	2.60	1.73
$t_y \times 10^3$ (kips)$^{-1}$	0.527	0.351	0.537	0.358	0.570	0.380
$t_r \times 10^3$ (kips)$^{-1}$	0.648	0.432	0.662	0.441	0.702	0.468
r_x/r_y	4.58		6.29		4.64	

[c] Shape is slender for compression with $F_y = 50$ ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W40

Shape		W40×											
		183^c				167^c				149^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**0.702**	0.467	**0.460**	0.306	**0.769**	0.512	**0.514**	0.342	**0.882**	0.587	**0.596**	0.396
	11	**0.823**	0.548	**0.485**	0.322	**0.909**	0.605	**0.548**	0.364	**1.05**	0.701	**0.644**	0.429
	12	**0.850**	0.565	**0.497**	0.330	**0.940**	0.626	**0.562**	0.374	**1.09**	0.726	**0.663**	0.441
	13	**0.880**	0.585	**0.509**	0.339	**0.976**	0.649	**0.577**	0.384	**1.14**	0.755	**0.682**	0.454
	14	**0.914**	0.608	**0.522**	0.348	**1.02**	0.676	**0.593**	0.395	**1.19**	0.789	**0.703**	0.468
	15	**0.953**	0.634	**0.536**	0.357	**1.06**	0.707	**0.610**	0.406	**1.24**	0.828	**0.725**	0.483
	16	**0.996**	0.663	**0.551**	0.367	**1.11**	0.742	**0.628**	0.418	**1.31**	0.872	**0.749**	0.498
	17	**1.05**	0.696	**0.566**	0.377	**1.17**	0.782	**0.648**	0.431	**1.39**	0.924	**0.774**	0.515
	18	**1.10**	0.734	**0.583**	0.388	**1.24**	0.828	**0.668**	0.444	**1.48**	0.984	**0.801**	0.533
	19	**1.17**	0.777	**0.600**	0.399	**1.32**	0.880	**0.690**	0.459	**1.58**	1.05	**0.830**	0.552
	20	**1.24**	0.826	**0.619**	0.412	**1.41**	0.941	**0.713**	0.474	**1.70**	1.13	**0.861**	0.573
	22	**1.43**	0.948	**0.659**	0.438	**1.65**	1.10	**0.764**	0.508	**2.02**	1.34	**0.930**	0.619
	24	**1.67**	1.11	**0.705**	0.469	**1.95**	1.30	**0.823**	0.548	**2.40**	1.60	**1.03**	0.683
	26	**1.96**	1.30	**0.762**	0.507	**2.29**	1.52	**0.921**	0.613	**2.82**	1.88	**1.18**	0.783
	28	**2.27**	1.51	**0.858**	0.571	**2.65**	1.76	**1.04**	0.692	**3.27**	2.17	**1.33**	0.887
	30	**2.61**	1.74	**0.955**	0.636	**3.05**	2.03	**1.16**	0.773	**3.75**	2.50	**1.49**	0.994
	32	**2.97**	1.97	**1.05**	0.702	**3.46**	2.31	**1.29**	0.855	**4.27**	2.84	**1.66**	1.10
	34	**3.35**	2.23	**1.15**	0.768	**3.91**	2.60	**1.41**	0.939	**4.82**	3.21	**1.82**	1.21
	36	**3.76**	2.50	**1.26**	0.836	**4.38**	2.92	**1.54**	1.02	**5.40**	3.59	**1.99**	1.33
	38	**4.18**	2.78	**1.36**	0.904	**4.89**	3.25	**1.67**	1.11	**6.02**	4.01	**2.17**	1.44
	40	**4.64**	3.09	**1.46**	0.972	**5.41**	3.60	**1.80**	1.19				

Other Constants and Properties

	183^c		167^c		149^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**4.03**	2.68	**4.69**	3.12	**5.74**	3.82
$t_y \times 10^3$ (kips)$^{-1}$	**0.626**	0.417	**0.678**	0.452	**0.761**	0.507
$t_r \times 10^3$ (kips)$^{-1}$	**0.770**	0.513	**0.834**	0.556	**0.936**	0.624
r_x/r_y	6.31		6.38		6.55	

[c] Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W36

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W36×											
		800^h				652^h				529^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.142	0.094	0.0976	0.0649	0.174	0.116	0.122	0.0815	0.214	0.142	0.153	0.102
	11	0.152	0.101	0.0976	0.0649	0.188	0.125	0.122	0.0815	0.232	0.154	0.153	0.102
	12	0.154	0.103	0.0976	0.0649	0.190	0.127	0.122	0.0815	0.235	0.157	0.153	0.102
	13	0.156	0.104	0.0976	0.0649	0.193	0.129	0.122	0.0815	0.239	0.159	0.153	0.102
	14	0.159	0.106	0.0976	0.0649	0.197	0.131	0.122	0.0815	0.244	0.162	0.153	0.102
	15	0.162	0.108	0.0977	0.0650	0.200	0.133	0.123	0.0817	0.248	0.165	0.154	0.102
	16	0.165	0.110	0.0982	0.0653	0.204	0.136	0.124	0.0823	0.253	0.169	0.155	0.103
	17	0.168	0.112	0.0987	0.0657	0.208	0.139	0.124	0.0828	0.259	0.172	0.157	0.104
	18	0.171	0.114	0.0992	0.0660	0.213	0.142	0.125	0.0833	0.265	0.176	0.158	0.105
	19	0.175	0.117	0.0997	0.0664	0.218	0.145	0.126	0.0839	0.272	0.181	0.159	0.106
	20	0.179	0.119	0.100	0.0667	0.223	0.149	0.127	0.0845	0.279	0.185	0.160	0.107
	22	0.188	0.125	0.101	0.0674	0.236	0.157	0.129	0.0856	0.294	0.196	0.163	0.109
	24	0.199	0.132	0.102	0.0682	0.250	0.166	0.130	0.0868	0.313	0.208	0.166	0.110
	26	0.211	0.140	0.104	0.0689	0.266	0.177	0.132	0.0880	0.334	0.222	0.169	0.112
	28	0.225	0.150	0.105	0.0697	0.284	0.189	0.134	0.0892	0.359	0.239	0.172	0.114
	30	0.241	0.160	0.106	0.0705	0.306	0.203	0.136	0.0905	0.387	0.258	0.175	0.117
	32	0.259	0.173	0.107	0.0713	0.330	0.220	0.138	0.0918	0.420	0.279	0.178	0.119
	34	0.280	0.187	0.108	0.0721	0.359	0.239	0.140	0.0932	0.458	0.305	0.182	0.121
	36	0.305	0.203	0.110	0.0730	0.392	0.261	0.142	0.0946	0.502	0.334	0.185	0.123
	38	0.332	0.221	0.111	0.0738	0.430	0.286	0.144	0.0960	0.554	0.369	0.189	0.126
	40	0.365	0.243	0.112	0.0747	0.475	0.316	0.147	0.0975	0.614	0.409	0.193	0.128
	42	0.402	0.268	0.114	0.0756	0.524	0.348	0.149	0.0990	0.677	0.450	0.197	0.131
	44	0.441	0.294	0.115	0.0766	0.575	0.382	0.151	0.101	0.743	0.494	0.201	0.134
	46	0.482	0.321	0.117	0.0775	0.628	0.418	0.154	0.102	0.812	0.540	0.205	0.137
	48	0.525	0.349	0.118	0.0785	0.684	0.455	0.156	0.104	0.884	0.588	0.210	0.140
	50	0.570	0.379	0.119	0.0795	0.742	0.494	0.159	0.106	0.960	0.638	0.215	0.143
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		0.479		0.319		0.613		0.408		0.785		0.522	
$t_y \times 10^3$ (kips)$^{-1}$		0.141		0.0940		0.174		0.116		0.213		0.142	
$t_r \times 10^3$ (kips)$^{-1}$		0.174		0.116		0.213		0.142		0.263		0.175	
r_x/r_y		3.93				3.95				4.00			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

W36

Shape		W36×											
		487^h				441^h				395^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**0.234**	0.155	**0.167**	0.111	**0.257**	0.171	**0.187**	0.124	**0.288**	0.192	**0.208**	0.139
	11	**0.253**	0.169	**0.167**	0.111	**0.279**	0.186	**0.187**	0.124	**0.313**	0.208	**0.208**	0.139
	12	**0.257**	0.171	**0.167**	0.111	**0.284**	0.189	**0.187**	0.124	**0.318**	0.212	**0.208**	0.139
	13	**0.262**	0.174	**0.167**	0.111	**0.288**	0.192	**0.187**	0.124	**0.324**	0.216	**0.208**	0.139
	14	**0.266**	0.177	**0.167**	0.111	**0.294**	0.196	**0.187**	0.124	**0.330**	0.220	**0.209**	0.139
	15	**0.272**	0.181	**0.169**	0.112	**0.300**	0.199	**0.189**	0.125	**0.337**	0.224	**0.211**	0.141
	16	**0.277**	0.185	**0.170**	0.113	**0.306**	0.204	**0.190**	0.127	**0.344**	0.229	**0.213**	0.142
	17	**0.284**	0.189	**0.172**	0.114	**0.313**	0.208	**0.192**	0.128	**0.352**	0.234	**0.216**	0.144
	18	**0.290**	0.193	**0.173**	0.115	**0.321**	0.213	**0.194**	0.129	**0.361**	0.240	**0.218**	0.145
	19	**0.298**	0.198	**0.175**	0.116	**0.329**	0.219	**0.196**	0.130	**0.371**	0.247	**0.221**	0.147
	20	**0.306**	0.203	**0.176**	0.117	**0.338**	0.225	**0.198**	0.132	**0.381**	0.253	**0.223**	0.148
	22	**0.323**	0.215	**0.180**	0.120	**0.358**	0.238	**0.202**	0.135	**0.404**	0.269	**0.228**	0.152
	24	**0.344**	0.229	**0.183**	0.122	**0.381**	0.254	**0.206**	0.137	**0.431**	0.287	**0.234**	0.155
	26	**0.368**	0.245	**0.187**	0.124	**0.408**	0.272	**0.211**	0.140	**0.462**	0.307	**0.239**	0.159
	28	**0.395**	0.263	**0.190**	0.127	**0.440**	0.293	**0.215**	0.143	**0.498**	0.331	**0.245**	0.163
	30	**0.427**	0.284	**0.194**	0.129	**0.476**	0.317	**0.220**	0.147	**0.540**	0.359	**0.251**	0.167
	32	**0.465**	0.309	**0.198**	0.132	**0.518**	0.345	**0.225**	0.150	**0.589**	0.392	**0.258**	0.171
	34	**0.508**	0.338	**0.202**	0.135	**0.567**	0.377	**0.231**	0.153	**0.646**	0.430	**0.265**	0.176
	36	**0.558**	0.371	**0.207**	0.138	**0.624**	0.415	**0.236**	0.157	**0.713**	0.474	**0.272**	0.181
	38	**0.617**	0.410	**0.211**	0.141	**0.693**	0.461	**0.242**	0.161	**0.792**	0.527	**0.279**	0.186
	40	**0.684**	0.455	**0.216**	0.144	**0.767**	0.511	**0.248**	0.165	**0.878**	0.584	**0.288**	0.191
	42	**0.754**	0.501	**0.221**	0.147	**0.846**	0.563	**0.255**	0.169	**0.968**	0.644	**0.296**	0.197
	44	**0.827**	0.550	**0.226**	0.150	**0.928**	0.618	**0.261**	0.174	**1.06**	0.707	**0.305**	0.203
	46	**0.904**	0.601	**0.232**	0.154	**1.01**	0.675	**0.268**	0.179	**1.16**	0.772	**0.315**	0.209
	48	**0.984**	0.655	**0.237**	0.158	**1.10**	0.735	**0.276**	0.184	**1.26**	0.841	**0.325**	0.216
	50	**1.07**	0.711	**0.243**	0.162	**1.20**	0.798	**0.284**	0.189	**1.37**	0.913	**0.336**	0.224

Other Constants and Properties

	487^h ASD	487^h LRFD	441^h ASD	441^h LRFD	395^h ASD	395^h LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	**0.865**	0.575	**0.968**	0.644	**1.10**	0.729
$t_y \times 10^3$ (kips)$^{-1}$	**0.233**	0.155	**0.257**	0.171	**0.288**	0.192
$t_r \times 10^3$ (kips)$^{-1}$	**0.287**	0.191	**0.315**	0.210	**0.354**	0.236
r_x/r_y	3.99		4.01		4.05	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W36

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W36×											
		361^h				330				302			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.315	0.210	0.230	0.153	0.344	0.229	0.253	0.168	0.376	0.250	0.278	0.185
	11	0.343	0.228	0.230	0.153	0.376	0.250	0.253	0.168	0.410	0.273	0.278	0.185
	12	0.349	0.232	0.230	0.153	0.382	0.254	0.253	0.168	0.417	0.278	0.278	0.185
	13	0.355	0.236	0.230	0.153	0.389	0.259	0.253	0.168	0.425	0.283	0.278	0.185
	14	0.362	0.241	0.231	0.154	0.397	0.264	0.254	0.169	0.433	0.288	0.280	0.186
	15	0.370	0.246	0.234	0.155	0.405	0.269	0.257	0.171	0.442	0.294	0.284	0.189
	16	0.378	0.251	0.236	0.157	0.414	0.275	0.260	0.173	0.452	0.301	0.287	0.191
	17	0.387	0.257	0.239	0.159	0.424	0.282	0.264	0.175	0.463	0.308	0.291	0.194
	18	0.397	0.264	0.242	0.161	0.435	0.289	0.267	0.178	0.475	0.316	0.295	0.196
	19	0.407	0.271	0.245	0.163	0.446	0.297	0.270	0.180	0.488	0.325	0.299	0.199
	20	0.419	0.279	0.248	0.165	0.459	0.305	0.274	0.182	0.502	0.334	0.303	0.202
	22	0.444	0.296	0.254	0.169	0.488	0.324	0.281	0.187	0.533	0.355	0.312	0.208
	24	0.474	0.316	0.260	0.173	0.521	0.347	0.289	0.192	0.570	0.379	0.321	0.214
	26	0.509	0.339	0.267	0.178	0.560	0.372	0.297	0.198	0.612	0.407	0.331	0.220
	28	0.550	0.366	0.275	0.183	0.605	0.402	0.306	0.204	0.662	0.440	0.341	0.227
	30	0.597	0.397	0.282	0.188	0.657	0.437	0.315	0.210	0.720	0.479	0.352	0.234
	32	0.652	0.434	0.290	0.193	0.718	0.478	0.325	0.216	0.787	0.524	0.364	0.242
	34	0.716	0.477	0.299	0.199	0.790	0.526	0.335	0.223	0.866	0.576	0.376	0.250
	36	0.791	0.526	0.308	0.205	0.873	0.581	0.346	0.230	0.958	0.637	0.390	0.259
	38	0.880	0.586	0.317	0.211	0.973	0.647	0.358	0.238	1.07	0.710	0.404	0.269
	40	0.976	0.649	0.328	0.218	1.08	0.717	0.371	0.247	1.18	0.787	0.419	0.279
	42	1.08	0.716	0.339	0.225	1.19	0.791	0.384	0.256	1.30	0.867	0.436	0.290
	44	1.18	0.785	0.350	0.233	1.30	0.868	0.399	0.265	1.43	0.952	0.457	0.304
	46	1.29	0.858	0.363	0.241	1.43	0.948	0.417	0.277	1.56	1.04	0.485	0.323
	48	1.40	0.935	0.376	0.250	1.55	1.03	0.441	0.293	1.70	1.13	0.514	0.342
	50	1.52	1.01	0.396	0.263	1.68	1.12	0.464	0.309	1.85	1.23	0.542	0.361

Other Constants and Properties

	361^h ASD	361^h LRFD	330 ASD	330 LRFD	302 ASD	302 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	1.22	0.809	1.34	0.894	1.48	0.984
$t_y \times 10^3$ (kips)$^{-1}$	0.315	0.210	0.344	0.229	0.375	0.250
$t_r \times 10^3$ (kips)$^{-1}$	0.387	0.258	0.423	0.282	0.462	0.308
r_x/r_y	4.05		4.05		4.03	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W36

Shape		W36×											
		282^c				262^c				256			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.404	0.269	0.299	0.199	0.441	0.294	0.324	0.215	0.443	0.295	0.343	0.228
	11	0.440	0.293	0.299	0.199	0.477	0.318	0.324	0.215	0.531	0.354	0.353	0.235
	12	0.447	0.298	0.299	0.199	0.485	0.323	0.324	0.215	0.550	0.366	0.360	0.239
	13	0.455	0.303	0.299	0.199	0.493	0.328	0.324	0.215	0.571	0.380	0.366	0.244
	14	0.465	0.309	0.302	0.201	0.502	0.334	0.327	0.218	0.595	0.396	0.374	0.249
	15	0.474	0.316	0.306	0.203	0.513	0.341	0.332	0.221	0.621	0.413	0.381	0.254
	16	0.485	0.323	0.310	0.206	0.525	0.349	0.337	0.224	0.651	0.433	0.389	0.259
	17	0.497	0.331	0.314	0.209	0.538	0.358	0.342	0.227	0.684	0.455	0.397	0.264
	18	0.510	0.339	0.319	0.212	0.552	0.367	0.347	0.231	0.720	0.479	0.405	0.270
	19	0.524	0.349	0.323	0.215	0.568	0.378	0.352	0.234	0.761	0.507	0.414	0.276
	20	0.539	0.359	0.328	0.218	0.585	0.389	0.357	0.238	0.807	0.537	0.423	0.282
	22	0.573	0.381	0.338	0.225	0.622	0.414	0.369	0.245	0.916	0.609	0.443	0.295
	24	0.613	0.408	0.348	0.232	0.666	0.443	0.381	0.253	1.05	0.699	0.465	0.309
	26	0.659	0.439	0.359	0.239	0.718	0.478	0.394	0.262	1.22	0.814	0.488	0.325
	28	0.713	0.475	0.371	0.247	0.778	0.518	0.408	0.271	1.42	0.944	0.515	0.342
	30	0.776	0.516	0.384	0.255	0.848	0.564	0.423	0.281	1.63	1.08	0.544	0.362
	32	0.850	0.565	0.397	0.264	0.930	0.619	0.439	0.292	1.85	1.23	0.581	0.386
	34	0.935	0.622	0.412	0.274	1.03	0.683	0.456	0.303	2.09	1.39	0.630	0.419
	36	1.04	0.690	0.427	0.284	1.14	0.759	0.475	0.316	2.35	1.56	0.679	0.452
	38	1.16	0.768	0.444	0.296	1.27	0.846	0.495	0.329	2.61	1.74	0.729	0.485
	40	1.28	0.852	0.462	0.308	1.41	0.937	0.517	0.344	2.90	1.93	0.778	0.517
	42	1.41	0.939	0.482	0.321	1.55	1.03	0.551	0.367	3.19	2.12	0.827	0.550
	44	1.55	1.03	0.514	0.342	1.70	1.13	0.590	0.392	3.50	2.33	0.876	0.583
	46	1.69	1.13	0.546	0.364	1.86	1.24	0.628	0.418				
	48	1.84	1.23	0.579	0.385	2.03	1.35	0.667	0.443				
	50	2.00	1.33	0.612	0.407	2.20	1.46	0.705	0.469				

Other Constants and Properties

	282^c ASD	282^c LRFD	262^c ASD	262^c LRFD	256 ASD	256 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	1.60	1.06	1.75	1.16	2.60	1.73
$t_y \times 10^3$ (kips)$^{-1}$	0.402	0.268	0.434	0.289	0.443	0.295
$t_r \times 10^3$ (kips)$^{-1}$	0.495	0.330	0.533	0.355	0.545	0.363
r_x/r_y	4.05		4.07		5.62	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W36

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W36×											
		247^c				232^c				231^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.474**	0.316	**0.346**	0.230	**0.497**	0.331	**0.381**	0.253	**0.512**	0.341	**0.370**	0.246
	11	**0.513**	0.341	**0.346**	0.230	**0.590**	0.393	**0.394**	0.262	**0.553**	0.368	**0.370**	0.246
	12	**0.521**	0.346	**0.346**	0.230	**0.612**	0.407	**0.402**	0.267	**0.562**	0.374	**0.370**	0.246
	13	**0.529**	0.352	**0.346**	0.230	**0.636**	0.423	**0.410**	0.273	**0.571**	0.38	**0.370**	0.246
	14	**0.539**	0.359	**0.350**	0.233	**0.662**	0.441	**0.419**	0.279	**0.582**	0.387	**0.375**	0.249
	15	**0.549**	0.366	**0.355**	0.236	**0.693**	0.461	**0.428**	0.284	**0.593**	0.394	**0.381**	0.253
	16	**0.561**	0.373	**0.360**	0.240	**0.726**	0.483	**0.437**	0.291	**0.605**	0.403	**0.387**	0.257
	17	**0.574**	0.382	**0.366**	0.243	**0.764**	0.508	**0.447**	0.297	**0.619**	0.412	**0.393**	0.261
	18	**0.588**	0.391	**0.372**	0.247	**0.806**	0.536	**0.457**	0.304	**0.634**	0.422	**0.399**	0.265
	19	**0.604**	0.402	**0.377**	0.251	**0.853**	0.568	**0.468**	0.311	**0.65**	0.433	**0.406**	0.270
	20	**0.622**	0.414	**0.384**	0.255	**0.906**	0.603	**0.479**	0.319	**0.668**	0.444	**0.412**	0.274
	22	**0.663**	0.441	**0.396**	0.264	**1.03**	0.686	**0.503**	0.335	**0.710**	0.473	**0.426**	0.284
	24	**0.710**	0.473	**0.410**	0.273	**1.19**	0.789	**0.530**	0.353	**0.762**	0.507	**0.442**	0.294
	26	**0.766**	0.510	**0.424**	0.282	**1.39**	0.922	**0.560**	0.372	**0.823**	0.547	**0.458**	0.305
	28	**0.831**	0.553	**0.440**	0.293	**1.61**	1.07	**0.593**	0.395	**0.893**	0.594	**0.475**	0.316
	30	**0.907**	0.603	**0.457**	0.304	**1.84**	1.23	**0.632**	0.421	**0.976**	0.650	**0.494**	0.329
	32	**0.995**	0.662	**0.475**	0.316	**2.10**	1.40	**0.692**	0.461	**1.07**	0.714	**0.515**	0.342
	34	**1.10**	0.731	**0.494**	0.329	**2.37**	1.58	**0.753**	0.501	**1.19**	0.790	**0.537**	0.357
	36	**1.22**	0.814	**0.516**	0.343	**2.66**	1.77	**0.813**	0.541	**1.32**	0.881	**0.561**	0.373
	38	**1.36**	0.907	**0.539**	0.359	**2.96**	1.97	**0.874**	0.581	**1.48**	0.982	**0.588**	0.391
	40	**1.51**	1.01	**0.569**	0.379	**3.28**	2.18	**0.934**	0.621	**1.64**	1.09	**0.631**	0.419
	42	**1.67**	1.11	**0.612**	0.407	**3.61**	2.41	**0.994**	0.662	**1.80**	1.20	**0.679**	0.452
	44	**1.83**	1.22	**0.655**	0.436					**1.98**	1.32	**0.728**	0.484
	46	**2.00**	1.33	**0.699**	0.465					**2.16**	1.44	**0.777**	0.517
	48	**2.18**	1.45	**0.743**	0.494					**2.36**	1.57	**0.827**	0.550
	50	**2.36**	1.57	**0.786**	0.523					**2.56**	1.70	**0.876**	0.583

Other Constants and Properties

	247^c ASD	247^c LRFD	232^c ASD	232^c LRFD	231^c ASD	231^c LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	**1.88**	1.25	**2.92**	1.94	**2.02**	1.35
$t_y \times 10^3$ (kips)$^{-1}$	**0.459**	0.306	**0.489**	0.326	**0.489**	0.326
$t_r \times 10^3$ (kips)$^{-1}$	**0.566**	0.377	**0.603**	0.402	**0.603**	0.402
r_x/r_y	4.06		5.65		4.07	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W36

Shape		W36×											
		210^c				194^c				182^c			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.556	0.370	0.428	0.285	0.618	0.411	0.464	0.309	0.669	0.445	0.496	0.330
	11	0.654	0.435	0.445	0.296	0.726	0.483	0.485	0.322	0.783	0.521	0.519	0.345
	12	0.679	0.451	0.455	0.302	0.749	0.498	0.496	0.330	0.808	0.538	0.531	0.353
	13	0.706	0.470	0.465	0.309	0.776	0.516	0.507	0.337	0.836	0.556	0.544	0.362
	14	0.737	0.490	0.475	0.316	0.806	0.536	0.519	0.345	0.869	0.578	0.557	0.371
	15	0.771	0.513	0.486	0.323	0.841	0.560	0.532	0.354	0.905	0.602	0.571	0.380
	16	0.810	0.539	0.498	0.331	0.884	0.588	0.545	0.363	0.947	0.630	0.586	0.390
	17	0.854	0.568	0.510	0.339	0.932	0.620	0.559	0.372	0.995	0.662	0.601	0.400
	18	0.902	0.600	0.523	0.348	0.986	0.656	0.574	0.382	1.05	0.700	0.617	0.411
	19	0.956	0.636	0.536	0.357	1.05	0.696	0.589	0.392	1.12	0.744	0.635	0.422
	20	1.02	0.677	0.550	0.366	1.11	0.741	0.605	0.403	1.19	0.792	0.653	0.434
	22	1.16	0.773	0.581	0.386	1.28	0.848	0.641	0.426	1.36	0.908	0.693	0.461
	24	1.34	0.894	0.615	0.409	1.48	0.984	0.681	0.453	1.58	1.05	0.738	0.491
	26	1.57	1.05	0.653	0.434	1.73	1.15	0.726	0.483	1.86	1.24	0.789	0.525
	28	1.83	1.21	0.697	0.463	2.01	1.34	0.784	0.522	2.15	1.43	0.867	0.577
	30	2.10	1.39	0.766	0.509	2.31	1.54	0.871	0.580	2.47	1.65	0.965	0.642
	32	2.38	1.59	0.841	0.560	2.63	1.75	0.960	0.638	2.81	1.87	1.06	0.708
	34	2.69	1.79	0.917	0.610	2.96	1.97	1.05	0.698	3.18	2.11	1.16	0.775
	36	3.02	2.01	0.994	0.661	3.32	2.21	1.14	0.757	3.56	2.37	1.27	0.842
	38	3.36	2.24	1.07	0.712	3.70	2.46	1.23	0.817	3.97	2.64	1.37	0.910
	40	3.73	2.48	1.15	0.763	4.10	2.73	1.32	0.877	4.40	2.93	1.47	0.978
	42	4.11	2.73	1.22	0.814	4.52	3.01	1.41	0.937	4.85	3.23	1.57	1.05

Other Constants and Properties

	210^c		194^c		182^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	3.33	2.22	3.65	2.43	3.93	2.61
$t_y \times 10^3$ (kips)$^{-1}$	0.540	0.360	0.585	0.390	0.623	0.415
$t_r \times 10^3$ (kips)$^{-1}$	0.663	0.442	0.720	0.480	0.765	0.510
r_x/r_y	5.66		5.70		5.69	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W36

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

$F_y = 50$ ksi

Shape		W36×											
		170^c				160^c				150^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.731	0.486	0.533	0.355	0.790	0.526	0.571	0.380	0.853	0.568	0.613	0.408
	11	0.855	0.569	0.559	0.372	0.925	0.615	0.601	0.400	0.999	0.665	0.648	0.431
	12	0.882	0.587	0.573	0.381	0.954	0.635	0.616	0.410	1.03	0.686	0.665	0.442
	13	0.913	0.607	0.587	0.390	0.987	0.657	0.632	0.420	1.07	0.710	0.683	0.454
	14	0.947	0.63	0.602	0.400	1.03	0.682	0.648	0.431	1.11	0.737	0.701	0.466
	15	0.987	0.657	0.617	0.411	1.07	0.711	0.666	0.443	1.16	0.769	0.721	0.480
	16	1.03	0.687	0.634	0.422	1.12	0.744	0.684	0.455	1.21	0.804	0.741	0.493
	17	1.08	0.721	0.651	0.433	1.17	0.781	0.704	0.468	1.27	0.845	0.763	0.508
	18	1.14	0.76	0.670	0.445	1.24	0.823	0.724	0.482	1.34	0.891	0.787	0.523
	19	1.21	0.804	0.689	0.458	1.31	0.871	0.746	0.496	1.42	0.945	0.811	0.540
	20	1.29	0.857	0.709	0.472	1.39	0.927	0.769	0.512	1.51	1.01	0.838	0.557
	22	1.48	0.984	0.755	0.502	1.60	1.07	0.821	0.546	1.74	1.16	0.896	0.596
	24	1.72	1.15	0.806	0.536	1.88	1.25	0.879	0.585	2.05	1.36	0.963	0.641
	26	2.02	1.34	0.864	0.575	2.20	1.47	0.952	0.634	2.40	1.60	1.06	0.707
	28	2.34	1.56	0.966	0.643	2.55	1.70	1.08	0.716	2.78	1.85	1.20	0.800
	30	2.69	1.79	1.08	0.717	2.93	1.95	1.20	0.799	3.20	2.13	1.35	0.895
	32	3.06	2.04	1.19	0.792	3.34	2.22	1.33	0.885	3.64	2.42	1.49	0.993
	34	3.46	2.30	1.31	0.868	3.77	2.51	1.46	0.972	4.10	2.73	1.64	1.09
	36	3.88	2.58	1.42	0.946	4.22	2.81	1.59	1.06	4.60	3.06	1.79	1.19
	38	4.32	2.87	1.54	1.02	4.70	3.13	1.73	1.15	5.13	3.41	1.95	1.29
	40	4.78	3.18	1.66	1.10	5.21	3.47	1.86	1.24	5.68	3.78	2.10	1.40
	42	5.28	3.51	1.77	1.18								

Other Constants and Properties

	170^c		160^c		150^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	4.25	2.83	4.61	3.07	5.02	3.34
$t_y \times 10^3$ (kips)$^{-1}$	0.666	0.444	0.708	0.472	0.753	0.502
$t_r \times 10^3$ (kips)$^{-1}$	0.819	0.546	0.872	0.581	0.927	0.618
r_x/r_y	5.73		5.76		5.79	

^{c} Shape is slender for compression with $F_y = 50$ ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W36–W33

Shape		W36×				W33×							
		135^c				387^h				354^h			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**0.971**	0.646	**0.700**	0.466	**0.293**	0.195	**0.228**	0.152	**0.321**	0.214	**0.251**	0.167
	11	**1.14**	0.761	**0.749**	0.498	**0.320**	0.213	**0.228**	0.152	**0.352**	0.234	**0.251**	0.167
	12	**1.18**	0.787	**0.769**	0.512	**0.326**	0.217	**0.228**	0.152	**0.358**	0.238	**0.251**	0.167
	13	**1.23**	0.816	**0.791**	0.526	**0.332**	0.221	**0.228**	0.152	**0.365**	0.243	**0.251**	0.167
	14	**1.28**	0.849	**0.814**	0.542	**0.339**	0.225	**0.230**	0.153	**0.372**	0.248	**0.253**	0.168
	15	**1.33**	0.887	**0.838**	0.558	**0.346**	0.230	**0.232**	0.155	**0.380**	0.253	**0.256**	0.170
	16	**1.40**	0.931	**0.864**	0.575	**0.354**	0.236	**0.235**	0.156	**0.389**	0.259	**0.259**	0.172
	17	**1.47**	0.981	**0.892**	0.593	**0.363**	0.241	**0.237**	0.158	**0.399**	0.266	**0.261**	0.174
	18	**1.56**	1.04	**0.921**	0.613	**0.372**	0.248	**0.239**	0.159	**0.410**	0.273	**0.264**	0.176
	19	**1.66**	1.10	**0.953**	0.634	**0.383**	0.255	**0.242**	0.161	**0.421**	0.280	**0.267**	0.178
	20	**1.77**	1.18	**0.986**	0.656	**0.394**	0.262	**0.244**	0.163	**0.434**	0.289	**0.270**	0.180
	22	**2.07**	1.37	**1.06**	0.706	**0.419**	0.279	**0.250**	0.166	**0.462**	0.308	**0.277**	0.184
	24	**2.45**	1.63	**1.15**	0.764	**0.449**	0.299	**0.255**	0.170	**0.495**	0.330	**0.283**	0.188
	26	**2.88**	1.91	**1.31**	0.872	**0.483**	0.322	**0.261**	0.174	**0.534**	0.355	**0.290**	0.193
	28	**3.34**	2.22	**1.49**	0.990	**0.524**	0.348	**0.267**	0.178	**0.579**	0.386	**0.298**	0.198
	30	**3.83**	2.55	**1.67**	1.11	**0.571**	0.380	**0.273**	0.182	**0.632**	0.421	**0.305**	0.203
	32	**4.36**	2.90	**1.86**	1.24	**0.626**	0.416	**0.280**	0.186	**0.694**	0.462	**0.313**	0.208
	34	**4.92**	3.27	**2.05**	1.36	**0.690**	0.459	**0.287**	0.191	**0.767**	0.510	**0.322**	0.214
	36	**5.51**	3.67	**2.24**	1.49	**0.766**	0.510	**0.294**	0.196	**0.854**	0.568	**0.331**	0.220
	38	**6.14**	4.09	**2.44**	1.62	**0.854**	0.568	**0.302**	0.201	**0.951**	0.633	**0.340**	0.226
	40					**0.946**	0.629	**0.310**	0.206	**1.05**	0.701	**0.350**	0.233
	42					**1.04**	0.694	**0.318**	0.212	**1.16**	0.773	**0.361**	0.240
	44					**1.14**	0.762	**0.327**	0.218	**1.28**	0.848	**0.372**	0.248
	46					**1.25**	0.832	**0.337**	0.224	**1.39**	0.927	**0.384**	0.256
	48					**1.36**	0.906	**0.347**	0.231	**1.52**	1.01	**0.397**	0.264
	50					**1.48**	0.984	**0.358**	0.238	**1.65**	1.10	**0.412**	0.274

Other Constants and Properties

	W36×135		W33×387		W33×354	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**5.97**	3.97	**1.14**	0.760	**1.26**	0.841
$t_y \times 10^3$ (kips)$^{-1}$	**0.839**	0.559	**0.293**	0.195	**0.321**	0.214
$t_r \times 10^3$ (kips)$^{-1}$	**1.03**	0.688	**0.360**	0.240	**0.395**	0.263
r_x/r_y	5.88		3.87		3.88	

^c Shape is slender for compression with F_y = 50 ksi.

^h Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

W33

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W33×											
		318				291				263			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.357	0.237	0.281	0.187	0.390	0.259	0.307	0.204	0.431	0.287	0.343	0.228
	11	0.391	0.260	0.281	0.187	0.428	0.285	0.307	0.204	0.474	0.315	0.343	0.228
	12	0.398	0.265	0.281	0.187	0.436	0.290	0.307	0.204	0.483	0.321	0.343	0.228
	13	0.406	0.270	0.281	0.187	0.445	0.296	0.307	0.204	0.492	0.328	0.343	0.228
	14	0.415	0.276	0.283	0.189	0.454	0.302	0.311	0.207	0.503	0.335	0.348	0.231
	15	0.424	0.282	0.287	0.191	0.464	0.309	0.315	0.210	0.515	0.342	0.352	0.234
	16	0.434	0.289	0.290	0.193	0.476	0.316	0.319	0.212	0.527	0.351	0.357	0.238
	17	0.445	0.296	0.294	0.196	0.488	0.325	0.323	0.215	0.541	0.360	0.362	0.241
	18	0.457	0.304	0.297	0.198	0.501	0.334	0.328	0.218	0.556	0.370	0.367	0.244
	19	0.470	0.313	0.301	0.200	0.516	0.343	0.332	0.221	0.573	0.381	0.373	0.248
	20	0.485	0.322	0.305	0.203	0.532	0.354	0.336	0.224	0.590	0.393	0.378	0.252
	22	0.517	0.344	0.313	0.208	0.568	0.378	0.346	0.230	0.631	0.420	0.390	0.259
	24	0.554	0.369	0.321	0.214	0.610	0.406	0.356	0.237	0.678	0.451	0.402	0.267
	26	0.599	0.398	0.330	0.220	0.659	0.439	0.366	0.244	0.733	0.488	0.415	0.276
	28	0.650	0.433	0.339	0.226	0.717	0.477	0.378	0.251	0.798	0.531	0.429	0.285
	30	0.710	0.473	0.349	0.232	0.785	0.522	0.390	0.259	0.875	0.582	0.443	0.295
	32	0.781	0.520	0.360	0.239	0.864	0.575	0.402	0.268	0.964	0.642	0.459	0.306
	34	0.864	0.575	0.371	0.247	0.958	0.637	0.416	0.277	1.07	0.712	0.476	0.317
	36	0.964	0.641	0.382	0.254	1.07	0.712	0.430	0.286	1.20	0.796	0.495	0.329
	38	1.07	0.715	0.395	0.263	1.19	0.793	0.446	0.297	1.33	0.887	0.514	0.342
	40	1.19	0.792	0.408	0.272	1.32	0.879	0.463	0.308	1.48	0.983	0.536	0.356
	42	1.31	0.873	0.423	0.281	1.46	0.969	0.481	0.320	1.63	1.08	0.562	0.374
	44	1.44	0.958	0.438	0.291	1.60	1.06	0.501	0.333	1.79	1.19	0.599	0.398
	46	1.57	1.05	0.455	0.302	1.75	1.16	0.531	0.353	1.95	1.30	0.635	0.423
	48	1.71	1.14	0.478	0.318	1.90	1.27	0.561	0.373	2.13	1.42	0.672	0.447
	50	1.86	1.24	0.503	0.335	2.06	1.37	0.590	0.393	2.31	1.54	0.709	0.472

Other Constants and Properties

	318		291		263	
$b_y \times 10^3$ (kip-ft)$^{-1}$	1.43	0.948	1.58	1.05	1.76	1.17
$t_y \times 10^3$ (kips)$^{-1}$	0.356	0.237	0.389	0.259	0.431	0.287
$t_r \times 10^3$ (kips)$^{-1}$	0.438	0.292	0.479	0.319	0.530	0.353
r_x/r_y	3.91		3.91		3.91	

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W33

Shape		W33×											
		241[c]				221[c]				201[c]			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**0.472**	0.314	**0.379**	0.252	**0.523**	0.348	**0.416**	0.277	**0.587**	0.391	**0.461**	0.307
	11	**0.519**	0.345	**0.379**	0.252	**0.569**	0.379	**0.416**	0.277	**0.640**	0.425	**0.461**	0.307
	12	**0.528**	0.351	**0.379**	0.252	**0.579**	0.385	**0.416**	0.277	**0.650**	0.433	**0.461**	0.307
	13	**0.539**	0.359	**0.380**	0.253	**0.590**	0.392	**0.418**	0.278	**0.662**	0.440	**0.464**	0.309
	14	**0.551**	0.366	**0.386**	0.257	**0.602**	0.400	**0.424**	0.282	**0.675**	0.449	**0.471**	0.314
	15	**0.564**	0.375	**0.391**	0.260	**0.616**	0.410	**0.431**	0.286	**0.689**	0.459	**0.479**	0.319
	16	**0.578**	0.385	**0.397**	0.264	**0.632**	0.420	**0.437**	0.291	**0.705**	0.469	**0.487**	0.324
	17	**0.594**	0.395	**0.403**	0.268	**0.649**	0.432	**0.444**	0.296	**0.723**	0.481	**0.495**	0.329
	18	**0.610**	0.406	**0.409**	0.272	**0.668**	0.444	**0.451**	0.300	**0.742**	0.494	**0.504**	0.335
	19	**0.629**	0.418	**0.416**	0.276	**0.688**	0.458	**0.459**	0.305	**0.763**	0.508	**0.512**	0.341
	20	**0.649**	0.432	**0.422**	0.281	**0.711**	0.473	**0.467**	0.310	**0.787**	0.524	**0.521**	0.347
	22	**0.694**	0.462	**0.436**	0.290	**0.761**	0.506	**0.483**	0.321	**0.844**	0.561	**0.541**	0.360
	24	**0.747**	0.497	**0.450**	0.300	**0.821**	0.546	**0.500**	0.333	**0.911**	0.606	**0.561**	0.373
	26	**0.810**	0.539	**0.466**	0.310	**0.891**	0.593	**0.519**	0.345	**0.990**	0.658	**0.583**	0.388
	28	**0.883**	0.588	**0.483**	0.321	**0.973**	0.647	**0.539**	0.358	**1.08**	0.720	**0.607**	0.404
	30	**0.970**	0.645	**0.501**	0.333	**1.07**	0.711	**0.560**	0.373	**1.19**	0.793	**0.633**	0.421
	32	**1.07**	0.713	**0.520**	0.346	**1.18**	0.787	**0.584**	0.388	**1.32**	0.879	**0.662**	0.440
	34	**1.19**	0.792	**0.541**	0.360	**1.32**	0.878	**0.609**	0.405	**1.48**	0.982	**0.693**	0.461
	36	**1.33**	0.888	**0.564**	0.375	**1.48**	0.984	**0.637**	0.424	**1.66**	1.10	**0.727**	0.484
	38	**1.49**	0.989	**0.589**	0.392	**1.65**	1.10	**0.667**	0.444	**1.84**	1.23	**0.781**	0.519
	40	**1.65**	1.10	**0.619**	0.412	**1.83**	1.21	**0.718**	0.478	**2.04**	1.36	**0.844**	0.562
	42	**1.82**	1.21	**0.663**	0.441	**2.01**	1.34	**0.772**	0.513	**2.25**	1.50	**0.908**	0.604
	44	**1.99**	1.33	**0.708**	0.471	**2.21**	1.47	**0.825**	0.549	**2.47**	1.65	**0.973**	0.647
	46	**2.18**	1.45	**0.753**	0.501	**2.41**	1.61	**0.878**	0.584	**2.70**	1.80	**1.04**	0.691
	48	**2.37**	1.58	**0.797**	0.531	**2.63**	1.75	**0.932**	0.620	**2.94**	1.96	**1.10**	0.734
	50	**2.57**	1.71	**0.842**	0.560	**2.85**	1.90	**0.986**	0.656	**3.19**	2.12	**1.17**	0.778

Other Constants and Properties

	241[c] ASD	241[c] LRFD	221[c] ASD	221[c] LRFD	201[c] ASD	201[c] LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	**1.96**	1.30	**2.17**	1.45	**2.42**	1.61
$t_y \times 10^3$ (kips)$^{-1}$	**0.470**	0.313	**0.512**	0.341	**0.563**	0.375
$t_r \times 10^3$ (kips)$^{-1}$	**0.578**	0.385	**0.630**	0.420	**0.693**	0.462
r_x/r_y	3.90		3.93		3.93	

[c] Shape is slender for compression with F_y = 50 ksi.

W33

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W33×											
		169^c				152^c				141^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.720	0.479	0.566	0.377	0.812	0.540	0.637	0.424	0.890	0.592	0.693	0.461
	11	0.851	0.567	0.594	0.396	0.960	0.638	0.672	0.447	1.05	0.701	0.735	0.489
	12	0.880	0.586	0.608	0.405	0.992	0.660	0.689	0.459	1.09	0.724	0.754	0.502
	13	0.913	0.607	0.623	0.414	1.03	0.684	0.707	0.470	1.13	0.752	0.774	0.515
	14	0.950	0.632	0.638	0.425	1.07	0.712	0.725	0.482	1.18	0.783	0.795	0.529
	15	0.992	0.660	0.654	0.435	1.12	0.744	0.745	0.495	1.23	0.818	0.818	0.544
	16	1.04	0.692	0.671	0.446	1.17	0.780	0.765	0.509	1.29	0.859	0.841	0.560
	17	1.10	0.730	0.689	0.458	1.24	0.822	0.787	0.523	1.36	0.905	0.866	0.576
	18	1.16	0.775	0.707	0.471	1.31	0.869	0.810	0.539	1.44	0.958	0.893	0.594
	19	1.24	0.825	0.727	0.484	1.39	0.926	0.834	0.555	1.53	1.02	0.921	0.613
	20	1.32	0.881	0.748	0.498	1.49	0.990	0.860	0.572	1.64	1.09	0.951	0.633
	22	1.52	1.01	0.794	0.528	1.72	1.14	0.917	0.610	1.91	1.27	1.02	0.677
	24	1.78	1.19	0.845	0.562	2.02	1.34	0.981	0.653	2.25	1.50	1.09	0.727
	26	2.09	1.39	0.904	0.601	2.37	1.58	1.06	0.708	2.64	1.76	1.21	0.807
	28	2.43	1.62	0.997	0.663	2.75	1.83	1.20	0.797	3.06	2.04	1.37	0.910
	30	2.79	1.85	1.10	0.735	3.16	2.10	1.33	0.887	3.51	2.34	1.53	1.01
	32	3.17	2.11	1.21	0.808	3.59	2.39	1.47	0.978	4.00	2.66	1.69	1.12
	34	3.58	2.38	1.32	0.881	4.06	2.70	1.61	1.07	4.51	3.00	1.85	1.23
	36	4.01	2.67	1.44	0.955	4.55	3.03	1.75	1.16	5.06	3.37	2.01	1.34
	38	4.47	2.97	1.55	1.03	5.07	3.37	1.89	1.26	5.64	3.75	2.18	1.45
	40	4.95	3.30	1.66	1.10	5.61	3.74	2.03	1.35	6.25	4.16	2.35	1.56

Other Constants and Properties

	169^c ASD	169^c LRFD	152^c ASD	152^c LRFD	141^c ASD	141^c LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	4.22	2.81	4.82	3.21	5.33	3.54
$t_y \times 10^3$ (kips)$^{-1}$	0.674	0.449	0.746	0.497	0.803	0.535
$t_r \times 10^3$ (kips)$^{-1}$	0.828	0.552	0.917	0.611	0.987	0.658
r_x/r_y	5.48		5.47		5.51	

[c] Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

W33–W30

Shape		W33×								W30×			
		130^c				118^c				391^h			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.982	0.653	0.763	0.508	1.11	0.739	0.858	0.571	0.290	0.193	0.246	0.163
	11	1.16	0.774	0.814	0.542	1.32	0.880	0.926	0.616	0.319	0.212	0.246	0.163
	12	1.20	0.801	0.836	0.557	1.37	0.911	0.952	0.633	0.325	0.216	0.246	0.163
	13	1.25	0.832	0.860	0.572	1.42	0.947	0.980	0.652	0.331	0.221	0.246	0.164
	14	1.30	0.867	0.884	0.588	1.49	0.988	1.01	0.672	0.339	0.225	0.248	0.165
	15	1.36	0.907	0.911	0.606	1.56	1.04	1.04	0.693	0.346	0.230	0.250	0.166
	16	1.43	0.952	0.938	0.624	1.64	1.09	1.08	0.715	0.355	0.236	0.252	0.168
	17	1.51	1.00	0.968	0.644	1.73	1.15	1.11	0.739	0.364	0.242	0.255	0.169
	18	1.60	1.06	0.999	0.665	1.84	1.22	1.15	0.765	0.374	0.249	0.257	0.171
	19	1.70	1.13	1.03	0.687	1.97	1.31	1.19	0.792	0.385	0.256	0.259	0.172
	20	1.82	1.21	1.07	0.711	2.11	1.41	1.24	0.822	0.397	0.264	0.262	0.174
	22	2.13	1.42	1.15	0.764	2.48	1.65	1.33	0.888	0.424	0.282	0.267	0.177
	24	2.52	1.68	1.24	0.825	2.96	1.97	1.48	0.982	0.456	0.303	0.272	0.181
	26	2.96	1.97	1.41	0.938	3.47	2.31	1.69	1.13	0.493	0.328	0.277	0.184
	28	3.43	2.28	1.59	1.06	4.02	2.68	1.92	1.28	0.536	0.357	0.282	0.188
	30	3.94	2.62	1.78	1.19	4.62	3.07	2.15	1.43	0.587	0.391	0.288	0.192
	32	4.48	2.98	1.97	1.31	5.26	3.50	2.39	1.59	0.647	0.430	0.294	0.196
	34	5.06	3.37	2.17	1.44	5.93	3.95	2.64	1.75	0.717	0.477	0.300	0.200
	36	5.67	3.78	2.37	1.58	6.65	4.43	2.88	1.92	0.802	0.533	0.307	0.204
	38	6.32	4.21	2.57	1.71	7.41	4.93	3.13	2.08	0.893	0.594	0.314	0.209
	40									0.990	0.658	0.321	0.213
	42									1.09	0.726	0.328	0.218
	44									1.20	0.797	0.336	0.224
	46									1.31	0.871	0.344	0.229
	48									1.43	0.948	0.353	0.235
	50									1.55	1.03	0.362	0.241

Other Constants and Properties

	W33×130^c		W33×118^c		W30×391^h	
$b_y \times 10^3$ (kip-ft)$^{-1}$	5.99	3.98	6.94	4.62	1.15	0.765
$t_y \times 10^3$ (kips)$^{-1}$	0.870	0.580	0.962	0.641	0.290	0.193
$t_r \times 10^3$ (kips)$^{-1}$	1.07	0.714	1.18	0.789	0.357	0.238
r_x/r_y	5.52		5.60		3.65	

^{c} Shape is slender for compression with F_y = 50 ksi.

^{h} Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W30

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W30×											
		357[h]				326[h]				292			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.318	0.212	0.270	0.180	0.349	0.232	0.299	0.199	0.389	0.259	0.336	0.224
	11	0.350	0.233	0.270	0.180	0.385	0.256	0.299	0.199	0.430	0.286	0.336	0.224
	12	0.357	0.237	0.270	0.180	0.392	0.261	0.299	0.199	0.438	0.291	0.336	0.224
	13	0.364	0.242	0.270	0.180	0.400	0.266	0.300	0.200	0.447	0.297	0.337	0.225
	14	0.372	0.247	0.273	0.182	0.409	0.272	0.303	0.202	0.457	0.304	0.341	0.227
	15	0.380	0.253	0.275	0.183	0.419	0.278	0.307	0.204	0.468	0.311	0.345	0.230
	16	0.390	0.259	0.278	0.185	0.429	0.286	0.310	0.206	0.480	0.319	0.349	0.232
	17	0.400	0.266	0.281	0.187	0.441	0.293	0.313	0.208	0.493	0.328	0.353	0.235
	18	0.412	0.274	0.284	0.189	0.454	0.302	0.316	0.211	0.508	0.338	0.358	0.238
	19	0.424	0.282	0.287	0.191	0.467	0.311	0.320	0.213	0.523	0.348	0.362	0.241
	20	0.437	0.291	0.290	0.193	0.482	0.321	0.323	0.215	0.540	0.360	0.366	0.244
	22	0.467	0.311	0.296	0.197	0.517	0.344	0.331	0.220	0.579	0.385	0.376	0.250
	24	0.503	0.334	0.302	0.201	0.557	0.370	0.338	0.225	0.624	0.415	0.385	0.256
	26	0.544	0.362	0.308	0.205	0.604	0.402	0.346	0.230	0.678	0.451	0.396	0.263
	28	0.593	0.395	0.315	0.210	0.659	0.439	0.355	0.236	0.741	0.493	0.406	0.270
	30	0.650	0.433	0.322	0.214	0.724	0.482	0.364	0.242	0.815	0.542	0.418	0.278
	32	0.718	0.478	0.330	0.219	0.801	0.533	0.373	0.248	0.902	0.600	0.430	0.286
	34	0.797	0.530	0.338	0.225	0.892	0.593	0.383	0.255	1.01	0.670	0.443	0.295
	36	0.892	0.594	0.346	0.230	1.00	0.665	0.393	0.261	1.13	0.751	0.456	0.304
	38	0.994	0.662	0.354	0.236	1.11	0.741	0.404	0.269	1.26	0.837	0.471	0.313
	40	1.10	0.733	0.363	0.242	1.23	0.821	0.415	0.276	1.39	0.927	0.486	0.323
	42	1.21	0.808	0.373	0.248	1.36	0.906	0.427	0.284	1.54	1.02	0.502	0.334
	44	1.33	0.887	0.383	0.255	1.49	0.994	0.440	0.293	1.69	1.12	0.520	0.346
	46	1.46	0.969	0.394	0.262	1.63	1.09	0.454	0.302	1.84	1.23	0.539	0.358
	48	1.59	1.06	0.405	0.269	1.78	1.18	0.468	0.311	2.01	1.33	0.564	0.375
	50	1.72	1.15	0.417	0.277	1.93	1.28	0.484	0.322	2.18	1.45	0.592	0.394
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		1.28		0.850		1.41		0.941		1.60		1.06	
$t_y \times 10^3$ (kips)$^{-1}$		0.318		0.212		0.348		0.232		0.389		0.259	
$t_r \times 10^3$ (kips)$^{-1}$		0.390		0.260		0.428		0.285		0.479		0.319	
r_x/r_y		3.65				3.67				3.69			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W30

Shape		W30×											
		261				235				211			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.434	0.289	0.378	0.251	0.483	0.321	0.421	0.280	0.537	0.357	0.474	0.316
	11	0.481	0.320	0.378	0.251	0.535	0.356	0.421	0.280	0.596	0.397	0.474	0.316
	12	0.491	0.326	0.378	0.251	0.546	0.363	0.421	0.280	0.608	0.405	0.474	0.316
	13	0.501	0.333	0.380	0.253	0.558	0.371	0.424	0.282	0.621	0.413	0.479	0.319
	14	0.513	0.341	0.385	0.256	0.571	0.380	0.430	0.286	0.636	0.423	0.486	0.323
	15	0.525	0.350	0.390	0.260	0.585	0.389	0.436	0.290	0.652	0.434	0.493	0.328
	16	0.539	0.359	0.395	0.263	0.601	0.400	0.442	0.294	0.670	0.446	0.501	0.333
	17	0.555	0.369	0.400	0.266	0.618	0.411	0.448	0.298	0.689	0.459	0.509	0.338
	18	0.571	0.380	0.406	0.270	0.637	0.424	0.455	0.303	0.711	0.473	0.517	0.344
	19	0.589	0.392	0.411	0.274	0.657	0.437	0.461	0.307	0.734	0.488	0.525	0.349
	20	0.609	0.405	0.417	0.277	0.680	0.452	0.468	0.312	0.759	0.505	0.533	0.355
	22	0.654	0.435	0.429	0.285	0.730	0.486	0.483	0.321	0.816	0.543	0.551	0.367
	24	0.707	0.470	0.441	0.293	0.790	0.525	0.498	0.331	0.884	0.588	0.571	0.380
	26	0.769	0.512	0.454	0.302	0.860	0.572	0.514	0.342	0.963	0.641	0.591	0.393
	28	0.843	0.561	0.468	0.311	0.943	0.628	0.532	0.354	1.06	0.704	0.613	0.408
	30	0.929	0.618	0.483	0.321	1.04	0.693	0.550	0.366	1.17	0.778	0.637	0.424
	32	1.03	0.687	0.499	0.332	1.16	0.771	0.570	0.379	1.30	0.866	0.663	0.441
	34	1.16	0.769	0.516	0.343	1.30	0.864	0.592	0.394	1.46	0.973	0.690	0.459
	36	1.30	0.862	0.534	0.355	1.46	0.969	0.615	0.409	1.64	1.09	0.721	0.479
	38	1.44	0.961	0.553	0.368	1.62	1.08	0.640	0.426	1.83	1.21	0.754	0.501
	40	1.60	1.06	0.574	0.382	1.80	1.20	0.667	0.444	2.02	1.35	0.803	0.535
	42	1.76	1.17	0.597	0.397	1.98	1.32	0.705	0.469	2.23	1.48	0.860	0.572
	44	1.94	1.29	0.625	0.416	2.18	1.45	0.749	0.498	2.45	1.63	0.916	0.609
	46	2.12	1.41	0.661	0.440	2.38	1.58	0.793	0.528	2.68	1.78	0.972	0.647
	48	2.30	1.53	0.697	0.463	2.59	1.72	0.838	0.557	2.91	1.94	1.03	0.684
	50	2.50	1.66	0.732	0.487	2.81	1.87	0.882	0.587	3.16	2.10	1.08	0.721

Other Constants and Properties

	261		235		211	
$b_y \times 10^3$ (kip-ft)$^{-1}$	1.82	1.21	2.04	1.35	2.30	1.53
$t_y \times 10^3$ (kips)$^{-1}$	0.434	0.289	0.482	0.321	0.536	0.357
$t_r \times 10^3$ (kips)$^{-1}$	0.534	0.356	0.593	0.395	0.660	0.440
r_x/r_y	3.71		3.70		3.70	

W30

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W30×											
		191^c				173^c				148^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.601	0.400	0.528	0.351	0.677	0.450	0.587	0.391	0.802	0.534	0.713	0.474
	11	0.661	0.439	0.528	0.351	0.743	0.495	0.587	0.391	0.988	0.657	0.765	0.509
	12	0.674	0.448	0.528	0.351	0.757	0.504	0.587	0.391	1.03	0.685	0.784	0.522
	13	0.689	0.458	0.534	0.355	0.772	0.514	0.596	0.396	1.08	0.719	0.804	0.535
	14	0.705	0.469	0.543	0.361	0.789	0.525	0.606	0.403	1.14	0.760	0.826	0.549
	15	0.723	0.481	0.551	0.367	0.808	0.537	0.616	0.410	1.21	0.806	0.849	0.565
	16	0.743	0.495	0.560	0.373	0.828	0.551	0.626	0.417	1.29	0.858	0.873	0.581
	17	0.765	0.509	0.569	0.379	0.851	0.566	0.637	0.424	1.38	0.917	0.898	0.597
	18	0.789	0.525	0.579	0.385	0.877	0.583	0.649	0.432	1.48	0.985	0.925	0.615
	19	0.815	0.542	0.589	0.392	0.907	0.603	0.660	0.439	1.60	1.06	0.953	0.634
	20	0.844	0.561	0.599	0.399	0.939	0.625	0.673	0.448	1.73	1.15	0.984	0.655
	22	0.908	0.604	0.621	0.413	1.01	0.674	0.698	0.465	2.05	1.36	1.05	0.699
	24	0.985	0.655	0.644	0.428	1.10	0.732	0.726	0.483	2.44	1.62	1.13	0.750
	26	1.08	0.716	0.669	0.445	1.20	0.801	0.756	0.503	2.86	1.91	1.24	0.827
	28	1.18	0.787	0.696	0.463	1.33	0.883	0.789	0.525	3.32	2.21	1.38	0.921
	30	1.31	0.871	0.725	0.482	1.47	0.980	0.825	0.549	3.81	2.54	1.53	1.02
	32	1.46	0.972	0.757	0.504	1.65	1.10	0.864	0.575	4.34	2.89	1.67	1.11
	34	1.64	1.09	0.792	0.527	1.86	1.24	0.907	0.603	4.90	3.26	1.81	1.21
	36	1.84	1.23	0.830	0.552	2.08	1.39	0.964	0.641	5.49	3.65	1.96	1.30
	38	2.05	1.37	0.886	0.590	2.32	1.54	1.05	0.696				
	40	2.28	1.51	0.955	0.635	2.57	1.71	1.13	0.752				
	42	2.51	1.67	1.02	0.681	2.83	1.89	1.21	0.807				
	44	2.75	1.83	1.09	0.727	3.11	2.07	1.30	0.864				
	46	3.01	2.00	1.16	0.773	3.40	2.26	1.38	0.920				
	48	3.28	2.18	1.23	0.819	3.70	2.46	1.47	0.976				
	50	3.56	2.37	1.30	0.865	4.02	2.67	1.55	1.03				

Other Constants and Properties

	191^c		173^c		148^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	2.58	1.72	2.90	1.93	5.24	3.49
$t_y \times 10^3$ (kips)$^{-1}$	0.593	0.395	0.654	0.436	0.767	0.511
$t_r \times 10^3$ (kips)$^{-1}$	0.729	0.486	0.804	0.536	0.944	0.629
r_x/r_y	3.70		3.71		5.44	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W30

Shape	W30×											
	132^c				124^c				116^c			
Design	$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
	(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**ASD**	LRFD	**ASD**	LRFD	**ASD**	LRFD	**ASD**	LRFD	**ASD**	LRFD	**ASD**	LRFD
0	**0.916**	0.609	**0.815**	0.542	**0.991**	0.659	**0.873**	0.581	**1.07**	0.713	**0.943**	0.627
11	**1.13**	0.750	**0.882**	0.587	**1.22**	0.811	**0.949**	0.631	**1.32**	0.880	**1.03**	0.686
12	**1.17**	0.781	**0.906**	0.603	**1.27**	0.845	**0.976**	0.649	**1.38**	0.918	**1.06**	0.706
13	**1.23**	0.818	**0.932**	0.620	**1.33**	0.885	**1.00**	0.668	**1.45**	0.962	**1.09**	0.728
14	**1.29**	0.860	**0.959**	0.638	**1.40**	0.931	**1.03**	0.688	**1.52**	1.01	**1.13**	0.750
15	**1.37**	0.913	**0.987**	0.657	**1.48**	0.984	**1.07**	0.710	**1.61**	1.07	**1.16**	0.775
16	**1.46**	0.974	**1.02**	0.677	**1.57**	1.05	**1.10**	0.733	**1.72**	1.14	**1.20**	0.801
17	**1.57**	1.04	**1.05**	0.699	**1.69**	1.12	**1.14**	0.757	**1.84**	1.23	**1.25**	0.828
18	**1.69**	1.12	**1.08**	0.722	**1.82**	1.21	**1.18**	0.783	**1.99**	1.32	**1.29**	0.858
19	**1.82**	1.21	**1.12**	0.746	**1.97**	1.31	**1.22**	0.811	**2.16**	1.44	**1.34**	0.890
20	**1.97**	1.31	**1.16**	0.773	**2.14**	1.42	**1.26**	0.840	**2.35**	1.56	**1.39**	0.924
22	**2.36**	1.57	**1.25**	0.831	**2.56**	1.70	**1.36**	0.907	**2.83**	1.88	**1.51**	1.00
24	**2.80**	1.87	**1.36**	0.905	**3.04**	2.02	**1.51**	1.01	**3.36**	2.24	**1.71**	1.13
26	**3.29**	2.19	**1.54**	1.03	**3.57**	2.38	**1.72**	1.14	**3.95**	2.63	**1.94**	1.29
28	**3.82**	2.54	**1.72**	1.15	**4.14**	2.75	**1.92**	1.28	**4.58**	3.05	**2.18**	1.45
30	**4.38**	2.92	**1.91**	1.27	**4.75**	3.16	**2.14**	1.42	**5.26**	3.50	**2.42**	1.61
32	**4.99**	3.32	**2.10**	1.39	**5.41**	3.60	**2.35**	1.56	**5.98**	3.98	**2.67**	1.78
34	**5.63**	3.75	**2.28**	1.52	**6.10**	4.06	**2.56**	1.71	**6.75**	4.49	**2.92**	1.94
36	**6.31**	4.20	**2.47**	1.64	**6.84**	4.55	**2.78**	1.85	**7.57**	5.04	**3.17**	2.11

Other Constants and Properties

	132^c		124^c		116^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**6.10**	4.06	**6.60**	4.39	**7.24**	4.82
$t_y \times 10^3$ (kips)$^{-1}$	**0.858**	0.572	**0.914**	0.609	**0.975**	0.650
$t_r \times 10^3$ (kips)$^{-1}$	**1.06**	0.704	**1.13**	0.750	**1.20**	0.800
r_x/r_y	5.42		5.43		5.48	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

W30

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W30×											
		108^c				99^c				90^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.17	0.781	1.03	0.685	1.31	0.869	1.14	0.760	1.49	0.991	1.26	0.838
	11	1.45	0.967	1.14	0.756	1.62	1.08	1.27	0.846	1.84	1.23	1.41	0.936
	12	1.52	1.01	1.17	0.779	1.70	1.13	1.31	0.874	1.92	1.28	1.45	0.968
	13	1.59	1.06	1.21	0.804	1.78	1.19	1.36	0.903	2.02	1.34	1.50	1.00
	14	1.68	1.12	1.25	0.831	1.88	1.25	1.41	0.935	2.13	1.41	1.56	1.04
	15	1.78	1.18	1.29	0.860	2.00	1.33	1.46	0.969	2.25	1.50	1.62	1.08
	16	1.90	1.26	1.34	0.890	2.14	1.42	1.51	1.01	2.40	1.60	1.68	1.12
	17	2.03	1.35	1.39	0.923	2.30	1.53	1.57	1.04	2.58	1.72	1.75	1.16
	18	2.20	1.47	1.44	0.959	2.49	1.66	1.63	1.09	2.79	1.85	1.82	1.21
	19	2.40	1.59	1.50	0.997	2.72	1.81	1.70	1.13	3.03	2.02	1.90	1.27
	20	2.62	1.74	1.56	1.04	2.99	1.99	1.78	1.18	3.33	2.21	1.99	1.32
	22	3.16	2.10	1.71	1.13	3.61	2.41	2.00	1.33	4.03	2.68	2.27	1.51
	24	3.76	2.50	1.97	1.31	4.30	2.86	2.31	1.54	4.79	3.19	2.64	1.76
	26	4.42	2.94	2.25	1.50	5.05	3.36	2.65	1.76	5.63	3.74	3.03	2.02
	28	5.12	3.41	2.53	1.69	5.86	3.90	2.99	1.99	6.52	4.34	3.43	2.28
	30	5.88	3.91	2.82	1.88	6.72	4.47	3.33	2.22	7.49	4.98	3.84	2.56
	32	6.69	4.45	3.12	2.07	7.65	5.09	3.69	2.46	8.52	5.67	4.26	2.84
	34	7.55	5.03	3.41	2.27	8.63	5.74	4.05	2.70	9.62	6.40	4.69	3.12

Other Constants and Properties

	108^c		99^c		90^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	8.12	5.40	9.23	6.14	10.3	6.83
$t_y \times 10^3$ (kips)$^{-1}$	1.05	0.701	1.15	0.764	1.26	0.843
$t_r \times 10^3$ (kips)$^{-1}$	1.29	0.862	1.41	0.940	1.56	1.04
r_x/r_y	5.53		5.57		5.60	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W27

Shape		W27×											
		539^h				368^h				336^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.210	0.140	0.189	0.125	0.309	0.206	0.287	0.191	0.338	0.225	0.315	0.210
	11	0.231	0.154	0.189	0.125	0.344	0.229	0.287	0.191	0.376	0.250	0.315	0.210
	12	0.235	0.157	0.189	0.125	0.351	0.233	0.287	0.191	0.383	0.255	0.315	0.210
	13	0.240	0.160	0.189	0.125	0.358	0.238	0.289	0.192	0.392	0.261	0.318	0.211
	14	0.245	0.163	0.190	0.126	0.367	0.244	0.291	0.194	0.402	0.267	0.320	0.213
	15	0.251	0.167	0.191	0.127	0.376	0.250	0.294	0.195	0.412	0.274	0.323	0.215
	16	0.257	0.171	0.192	0.128	0.386	0.257	0.296	0.197	0.423	0.282	0.326	0.217
	17	0.264	0.176	0.193	0.128	0.398	0.265	0.299	0.199	0.436	0.290	0.329	0.219
	18	0.271	0.181	0.194	0.129	0.410	0.273	0.301	0.200	0.450	0.299	0.332	0.221
	19	0.279	0.186	0.195	0.130	0.423	0.282	0.304	0.202	0.465	0.309	0.336	0.223
	20	0.288	0.192	0.196	0.131	0.438	0.291	0.306	0.204	0.481	0.320	0.339	0.225
	22	0.308	0.205	0.198	0.132	0.471	0.313	0.312	0.207	0.518	0.345	0.345	0.230
	24	0.331	0.220	0.201	0.134	0.510	0.340	0.317	0.211	0.562	0.374	0.352	0.234
	26	0.358	0.238	0.203	0.135	0.557	0.370	0.323	0.215	0.614	0.408	0.360	0.239
	28	0.390	0.260	0.206	0.137	0.611	0.407	0.329	0.219	0.675	0.449	0.367	0.244
	30	0.428	0.285	0.208	0.139	0.676	0.450	0.335	0.223	0.748	0.498	0.375	0.249
	32	0.472	0.314	0.211	0.140	0.753	0.501	0.342	0.227	0.835	0.556	0.383	0.255
	34	0.524	0.348	0.213	0.142	0.847	0.563	0.349	0.232	0.940	0.626	0.391	0.260
	36	0.586	0.390	0.216	0.144	0.949	0.632	0.356	0.237	1.05	0.702	0.400	0.266
	38	0.653	0.435	0.219	0.146	1.06	0.704	0.363	0.241	1.17	0.782	0.409	0.272
	40	0.724	0.481	0.222	0.148	1.17	0.780	0.371	0.247	1.30	0.866	0.419	0.279
	42	0.798	0.531	0.225	0.149	1.29	0.860	0.378	0.252	1.44	0.955	0.429	0.286
	44	0.876	0.583	0.228	0.151	1.42	0.944	0.387	0.257	1.58	1.05	0.440	0.293
	46	0.957	0.637	0.231	0.154	1.55	1.03	0.395	0.263	1.72	1.15	0.451	0.300
	48	1.04	0.693	0.234	0.156	1.69	1.12	0.404	0.269	1.87	1.25	0.463	0.308
	50	1.13	0.752	0.237	0.158	1.83	1.22	0.414	0.275	2.03	1.35	0.475	0.316

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	0.815	0.542	1.28	0.850	1.41	0.941
$t_y \times 10^3$ (kips)$^{-1}$	0.210	0.140	0.309	0.206	0.338	0.225
$t_r \times 10^3$ (kips)$^{-1}$	0.258	0.172	0.380	0.253	0.414	0.276
r_x/r_y	3.48		3.51		3.51	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W27

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W27×											
		307^h				281				258			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.369	0.246	0.346	0.230	0.403	0.268	0.381	0.253	0.440	0.292	0.418	0.278
	11	0.412	0.274	0.346	0.230	0.450	0.300	0.381	0.253	0.492	0.327	0.418	0.278
	12	0.421	0.280	0.346	0.230	0.460	0.306	0.381	0.253	0.503	0.335	0.419	0.279
	13	0.431	0.286	0.349	0.232	0.470	0.313	0.385	0.256	0.515	0.342	0.424	0.282
	14	0.441	0.294	0.353	0.235	0.482	0.321	0.389	0.259	0.528	0.351	0.429	0.285
	15	0.453	0.301	0.356	0.237	0.495	0.330	0.393	0.262	0.542	0.361	0.434	0.289
	16	0.466	0.310	0.360	0.239	0.510	0.339	0.397	0.264	0.558	0.371	0.439	0.292
	17	0.480	0.319	0.364	0.242	0.525	0.349	0.402	0.267	0.576	0.383	0.444	0.295
	18	0.495	0.330	0.367	0.244	0.542	0.361	0.406	0.270	0.595	0.396	0.450	0.299
	19	0.512	0.341	0.371	0.247	0.561	0.373	0.411	0.273	0.616	0.410	0.455	0.303
	20	0.531	0.353	0.375	0.250	0.581	0.387	0.416	0.277	0.638	0.425	0.461	0.307
	22	0.573	0.381	0.383	0.255	0.628	0.418	0.425	0.283	0.690	0.459	0.473	0.315
	24	0.622	0.414	0.392	0.261	0.683	0.454	0.436	0.290	0.752	0.500	0.485	0.323
	26	0.681	0.453	0.401	0.267	0.749	0.498	0.447	0.297	0.826	0.549	0.498	0.332
	28	0.751	0.500	0.410	0.273	0.827	0.550	0.458	0.305	0.913	0.608	0.512	0.341
	30	0.835	0.555	0.420	0.279	0.919	0.612	0.470	0.312	1.02	0.677	0.527	0.351
	32	0.934	0.621	0.430	0.286	1.03	0.685	0.482	0.321	1.14	0.761	0.543	0.361
	34	1.05	0.701	0.441	0.293	1.16	0.774	0.495	0.330	1.29	0.859	0.559	0.372
	36	1.18	0.786	0.452	0.301	1.30	0.867	0.509	0.339	1.45	0.963	0.577	0.384
	38	1.32	0.876	0.464	0.309	1.45	0.966	0.524	0.349	1.61	1.07	0.595	0.396
	40	1.46	0.970	0.476	0.317	1.61	1.07	0.540	0.359	1.79	1.19	0.615	0.409
	42	1.61	1.07	0.490	0.326	1.77	1.18	0.556	0.370	1.97	1.31	0.637	0.424
	44	1.76	1.17	0.504	0.335	1.95	1.30	0.574	0.382	2.16	1.44	0.660	0.439
	46	1.93	1.28	0.518	0.345	2.13	1.42	0.593	0.394	2.36	1.57	0.685	0.456
	48	2.10	1.40	0.534	0.355	2.32	1.54	0.613	0.408	2.57	1.71	0.720	0.479
	50	2.28	1.52	0.551	0.366	2.51	1.67	0.638	0.424	2.79	1.86	0.755	0.502

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	1.57	1.04	1.73	1.15	1.91	1.27
$t_y \times 10^3$ (kips)$^{-1}$	0.369	0.246	0.402	0.268	0.438	0.292
$t_r \times 10^3$ (kips)$^{-1}$	0.455	0.303	0.495	0.330	0.540	0.360
r_x/r_y	3.52		3.54		3.54	

^{h} Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W27

Shape		W27×											
		235				217				194			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.481	0.320	0.461	0.307	0.522	0.347	0.501	0.333	0.584	0.388	0.565	0.376
	11	0.540	0.359	0.461	0.307	0.586	0.390	0.501	0.333	0.657	0.437	0.565	0.376
	12	0.552	0.367	0.463	0.308	0.599	0.399	0.503	0.335	0.672	0.447	0.568	0.378
	13	0.565	0.376	0.469	0.312	0.613	0.408	0.510	0.339	0.688	0.458	0.576	0.383
	14	0.580	0.386	0.475	0.316	0.629	0.419	0.517	0.344	0.706	0.470	0.584	0.389
	15	0.596	0.397	0.481	0.320	0.647	0.431	0.524	0.348	0.727	0.483	0.593	0.395
	16	0.614	0.408	0.487	0.324	0.667	0.444	0.531	0.353	0.749	0.498	0.602	0.401
	17	0.633	0.421	0.494	0.328	0.688	0.458	0.538	0.358	0.773	0.514	0.612	0.407
	18	0.655	0.436	0.500	0.333	0.711	0.473	0.546	0.363	0.800	0.532	0.621	0.413
	19	0.678	0.451	0.507	0.337	0.737	0.490	0.554	0.369	0.829	0.552	0.631	0.420
	20	0.704	0.468	0.514	0.342	0.765	0.509	0.562	0.374	0.861	0.573	0.641	0.427
	22	0.762	0.507	0.529	0.352	0.829	0.551	0.579	0.385	0.935	0.622	0.663	0.441
	24	0.832	0.553	0.544	0.362	0.905	0.602	0.598	0.398	1.02	0.680	0.686	0.456
	26	0.915	0.609	0.560	0.373	0.996	0.662	0.617	0.410	1.13	0.750	0.711	0.473
	28	1.01	0.674	0.578	0.384	1.10	0.734	0.637	0.424	1.25	0.833	0.737	0.490
	30	1.13	0.753	0.596	0.397	1.23	0.821	0.660	0.439	1.40	0.932	0.766	0.509
	32	1.28	0.848	0.616	0.410	1.39	0.926	0.683	0.455	1.58	1.05	0.797	0.530
	34	1.44	0.958	0.637	0.424	1.57	1.04	0.709	0.471	1.79	1.19	0.830	0.552
	36	1.61	1.07	0.660	0.439	1.76	1.17	0.736	0.490	2.00	1.33	0.867	0.577
	38	1.80	1.20	0.684	0.455	1.96	1.31	0.766	0.509	2.23	1.49	0.906	0.603
	40	1.99	1.33	0.710	0.472	2.17	1.45	0.798	0.531	2.48	1.65	0.969	0.644
	42	2.20	1.46	0.738	0.491	2.40	1.59	0.842	0.560	2.73	1.82	1.03	0.687
	44	2.41	1.60	0.775	0.516	2.63	1.75	0.892	0.594	2.99	1.99	1.10	0.729
	46	2.64	1.75	0.818	0.544	2.87	1.91	0.942	0.627	3.27	2.18	1.16	0.771
	48	2.87	1.91	0.861	0.573	3.13	2.08	0.992	0.660	3.56	2.37	1.22	0.813
	50	3.11	2.07	0.903	0.601	3.40	2.26	1.04	0.693	3.87	2.57	1.29	0.855

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	2.12	1.41	2.31	1.54	2.62	1.74
$t_y \times 10^3$ (kips)$^{-1}$	0.480	0.320	0.521	0.347	0.582	0.388
$t_r \times 10^3$ (kips)$^{-1}$	0.591	0.394	0.641	0.427	0.717	0.478
r_x/r_y	3.54		3.55		3.56	

W27

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W27×											
		178				161^c				146^c			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.636	0.423	0.625	0.416	0.704	0.468	0.692	0.460	0.793	0.528	0.768	0.511
	11	0.718	0.478	0.625	0.416	0.793	0.528	0.692	0.460	0.884	0.588	0.768	0.511
	12	0.735	0.489	0.630	0.419	0.811	0.540	0.698	0.465	0.903	0.601	0.777	0.517
	13	0.753	0.501	0.640	0.426	0.832	0.554	0.710	0.472	0.924	0.615	0.791	0.526
	14	0.774	0.515	0.650	0.433	0.855	0.569	0.722	0.480	0.947	0.630	0.805	0.535
	15	0.796	0.530	0.661	0.440	0.881	0.586	0.734	0.489	0.976	0.649	0.819	0.545
	16	0.821	0.546	0.671	0.447	0.909	0.604	0.747	0.497	1.01	0.670	0.835	0.555
	17	0.849	0.565	0.683	0.454	0.939	0.625	0.761	0.506	1.04	0.693	0.850	0.566
	18	0.879	0.585	0.694	0.462	0.973	0.647	0.774	0.515	1.08	0.719	0.867	0.577
	19	0.912	0.607	0.706	0.470	1.01	0.672	0.789	0.525	1.12	0.746	0.884	0.588
	20	0.948	0.631	0.719	0.478	1.05	0.699	0.804	0.535	1.17	0.777	0.901	0.600
	22	1.03	0.686	0.745	0.496	1.14	0.761	0.835	0.556	1.27	0.847	0.939	0.625
	24	1.13	0.752	0.773	0.514	1.25	0.835	0.869	0.578	1.40	0.931	0.980	0.652
	26	1.25	0.831	0.803	0.535	1.39	0.924	0.906	0.603	1.55	1.03	1.02	0.681
	28	1.39	0.925	0.836	0.556	1.55	1.03	0.946	0.629	1.73	1.15	1.07	0.714
	30	1.56	1.04	0.872	0.580	1.74	1.16	0.990	0.659	1.95	1.30	1.13	0.749
	32	1.77	1.18	0.911	0.606	1.98	1.31	1.04	0.691	2.22	1.48	1.19	0.789
	34	2.00	1.33	0.953	0.634	2.23	1.48	1.09	0.726	2.51	1.67	1.27	0.843
	36	2.24	1.49	1.00	0.665	2.50	1.66	1.17	0.780	2.81	1.87	1.38	0.918
	38	2.50	1.66	1.07	0.715	2.79	1.85	1.27	0.843	3.13	2.08	1.49	0.994
	40	2.76	1.84	1.15	0.766	3.09	2.05	1.36	0.906	3.47	2.31	1.61	1.07
	42	3.05	2.03	1.23	0.818	3.40	2.26	1.46	0.969	3.82	2.54	1.73	1.15
	44	3.35	2.23	1.31	0.870	3.73	2.49	1.55	1.03	4.20	2.79	1.84	1.22
	46	3.66	2.43	1.39	0.922	4.08	2.72	1.65	1.10	4.59	3.05	1.96	1.30
	48	3.98	2.65	1.46	0.974	4.44	2.96	1.74	1.16	5.00	3.32	2.07	1.38
	50	4.32	2.87	1.54	1.03	4.82	3.21	1.84	1.22	5.42	3.61	2.19	1.46
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		2.92		1.94		3.27		2.17		3.65		2.43	
$t_y \times 10^3$ (kips)$^{-1}$		0.635		0.423		0.701		0.467		0.773		0.515	
$t_r \times 10^3$ (kips)$^{-1}$		0.782		0.521		0.863		0.575		0.951		0.634	
r_x/r_y		3.57				3.56				3.59			

[c] Shape is slender for compression with F_y = 50 ksi.

F_y = 50 ksi

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

W27

Shape	W27×											
	129^c				114^c				102^c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending												
0	0.910	0.605	0.902	0.600	1.05	0.698	1.04	0.691	1.21	0.803	1.17	0.777
11	1.15	0.762	0.975	0.649	1.31	0.874	1.13	0.754	1.51	1.00	1.28	0.854
12	1.20	0.801	1.00	0.666	1.38	0.915	1.17	0.776	1.58	1.05	1.32	0.880
13	1.27	0.845	1.03	0.684	1.45	0.964	1.20	0.798	1.66	1.10	1.36	0.907
14	1.35	0.896	1.06	0.703	1.54	1.02	1.24	0.822	1.75	1.17	1.41	0.936
15	1.43	0.954	1.09	0.723	1.64	1.09	1.27	0.848	1.86	1.24	1.45	0.967
16	1.53	1.02	1.12	0.744	1.76	1.17	1.32	0.875	1.99	1.33	1.50	1.00
17	1.65	1.09	1.15	0.766	1.89	1.26	1.36	0.904	2.15	1.43	1.56	1.03
18	1.77	1.18	1.19	0.790	2.04	1.36	1.41	0.935	2.33	1.55	1.61	1.07
19	1.92	1.28	1.23	0.816	2.22	1.47	1.46	0.969	2.53	1.68	1.67	1.11
20	2.09	1.39	1.27	0.843	2.42	1.61	1.51	1.00	2.77	1.84	1.74	1.16
22	2.51	1.67	1.36	0.903	2.91	1.94	1.63	1.08	3.34	2.22	1.89	1.26
24	2.99	1.99	1.46	0.971	3.46	2.30	1.81	1.20	3.98	2.65	2.16	1.44
26	3.50	2.33	1.63	1.09	4.06	2.70	2.04	1.36	4.67	3.10	2.45	1.63
28	4.06	2.70	1.81	1.21	4.71	3.14	2.28	1.52	5.41	3.60	2.74	1.82
30	4.67	3.10	1.99	1.33	5.41	3.60	2.52	1.68	6.21	4.13	3.04	2.02
32	5.31	3.53	2.18	1.45	6.16	4.10	2.76	1.84	7.07	4.70	3.34	2.22
34	5.99	3.99	2.36	1.57	6.95	4.62	3.00	2.00	7.98	5.31	3.64	2.42
36	6.72	4.47	2.54	1.69	7.79	5.18	3.24	2.16				

Other Constants and Properties

	129^c		114^c		102^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	6.19	4.12	7.23	4.81	8.21	5.46
$t_y \times 10^3$ (kips)$^{-1}$	0.881	0.587	0.995	0.663	1.11	0.740
$t_r \times 10^3$ (kips)$^{-1}$	1.08	0.723	1.22	0.816	1.37	0.911
r_x/r_y	5.07		5.05		5.12	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

W27–W24

F_y = 50 ksi

Shape		W27×								W24×			
		94^c				84^c				370^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.33	0.887	1.28	0.853	1.53	1.02	1.46	0.971	0.306	0.204	0.315	0.210
	11	1.67	1.11	1.42	0.944	1.92	1.28	1.63	1.09	0.345	0.230	0.315	0.210
	12	1.75	1.16	1.46	0.974	2.01	1.34	1.69	1.12	0.353	0.235	0.316	0.210
	13	1.84	1.22	1.51	1.01	2.12	1.41	1.75	1.16	0.362	0.241	0.319	0.212
	14	1.94	1.29	1.56	1.04	2.24	1.49	1.81	1.20	0.372	0.247	0.321	0.213
	15	2.07	1.37	1.62	1.08	2.39	1.59	1.88	1.25	0.382	0.254	0.323	0.215
	16	2.21	1.47	1.67	1.11	2.55	1.70	1.95	1.30	0.394	0.262	0.326	0.217
	17	2.38	1.58	1.74	1.16	2.75	1.83	2.03	1.35	0.407	0.271	0.328	0.218
	18	2.58	1.72	1.80	1.20	2.99	1.99	2.11	1.41	0.422	0.280	0.330	0.220
	19	2.81	1.87	1.88	1.25	3.28	2.18	2.20	1.47	0.437	0.291	0.333	0.221
	20	3.08	2.05	1.96	1.30	3.61	2.40	2.30	1.53	0.454	0.302	0.335	0.223
	22	3.73	2.48	2.17	1.44	4.37	2.91	2.64	1.76	0.494	0.328	0.340	0.226
	24	4.44	2.95	2.50	1.66	5.20	3.46	3.06	2.03	0.540	0.359	0.346	0.230
	26	5.21	3.47	2.84	1.89	6.11	4.06	3.49	2.32	0.596	0.397	0.351	0.234
	28	6.04	4.02	3.19	2.12	7.08	4.71	3.93	2.61	0.663	0.441	0.357	0.237
	30	6.94	4.62	3.55	2.36	8.13	5.41	4.38	2.91	0.743	0.495	0.363	0.241
	32	7.89	5.25	3.91	2.60	9.25	6.15	4.84	3.22	0.842	0.560	0.369	0.245
	34	8.91	5.93	4.27	2.84	10.4	6.95	5.30	3.53	0.950	0.632	0.375	0.249
	36									1.07	0.709	0.381	0.254
	38									1.19	0.790	0.388	0.258
	40									1.32	0.875	0.395	0.263
	42									1.45	0.965	0.402	0.267
	44									1.59	1.06	0.409	0.272
	46									1.74	1.16	0.417	0.277
	48									1.89	1.26	0.425	0.283
	50									2.05	1.37	0.433	0.288
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		9.18		6.11		10.7		7.14		1.33		0.888	
$t_y \times 10^3$ (kips)$^{-1}$		1.21		0.804		1.35		0.898		0.306		0.204	
$t_r \times 10^3$ (kips)$^{-1}$		1.48		0.989		1.66		1.1		0.377		0.251	
r_x/r_y		5.14				5.17				3.39			

^{c} Shape is slender for compression with F_y = 50 ksi.

^{h} Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W24

Shape		W24×											
		335^h				306^h				279^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.339	0.226	0.349	0.232	0.372	0.248	0.386	0.257	0.407	0.271	0.427	0.284
	11	0.383	0.255	0.349	0.232	0.421	0.280	0.386	0.257	0.462	0.308	0.427	0.284
	12	0.392	0.261	0.351	0.233	0.431	0.287	0.389	0.259	0.474	0.315	0.430	0.286
	13	0.403	0.268	0.354	0.235	0.443	0.294	0.392	0.261	0.486	0.324	0.434	0.289
	14	0.414	0.275	0.357	0.237	0.455	0.303	0.396	0.263	0.500	0.333	0.438	0.292
	15	0.426	0.283	0.359	0.239	0.469	0.312	0.399	0.266	0.516	0.343	0.443	0.294
	16	0.439	0.292	0.362	0.241	0.484	0.322	0.403	0.268	0.533	0.354	0.447	0.297
	17	0.454	0.302	0.365	0.243	0.501	0.333	0.406	0.270	0.551	0.367	0.451	0.300
	18	0.471	0.313	0.368	0.245	0.519	0.345	0.410	0.273	0.572	0.381	0.456	0.303
	19	0.489	0.325	0.372	0.247	0.539	0.359	0.414	0.275	0.595	0.396	0.461	0.306
	20	0.508	0.338	0.375	0.249	0.561	0.373	0.418	0.278	0.619	0.412	0.465	0.310
	22	0.553	0.368	0.381	0.254	0.612	0.407	0.426	0.283	0.676	0.450	0.475	0.316
	24	0.607	0.404	0.388	0.258	0.673	0.448	0.434	0.289	0.745	0.496	0.485	0.323
	26	0.671	0.447	0.395	0.263	0.745	0.496	0.443	0.294	0.827	0.550	0.496	0.330
	28	0.749	0.498	0.402	0.267	0.833	0.554	0.451	0.300	0.926	0.616	0.507	0.337
	30	0.842	0.560	0.409	0.272	0.939	0.624	0.461	0.307	1.05	0.696	0.519	0.345
	32	0.956	0.636	0.417	0.277	1.07	0.710	0.470	0.313	1.19	0.792	0.531	0.353
	34	1.08	0.718	0.425	0.283	1.20	0.801	0.481	0.320	1.34	0.894	0.544	0.362
	36	1.21	0.805	0.433	0.288	1.35	0.899	0.491	0.327	1.51	1.00	0.557	0.371
	38	1.35	0.897	0.442	0.294	1.50	1.00	0.502	0.334	1.68	1.12	0.571	0.380
	40	1.49	0.993	0.451	0.300	1.67	1.11	0.514	0.342	1.86	1.24	0.586	0.390
	42	1.65	1.10	0.460	0.306	1.84	1.22	0.526	0.350	2.05	1.36	0.602	0.400
	44	1.81	1.20	0.470	0.313	2.02	1.34	0.538	0.358	2.25	1.50	0.618	0.411
	46	1.97	1.31	0.480	0.320	2.21	1.47	0.552	0.367	2.46	1.64	0.635	0.423
	48	2.15	1.43	0.491	0.327	2.40	1.60	0.565	0.376	2.68	1.78	0.654	0.435
	50	2.33	1.55	0.502	0.334	2.61	1.73	0.580	0.386	2.91	1.93	0.673	0.448

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	1.50	0.996	1.66	1.11	1.85	1.23
$t_y \times 10^3$ (kips)$^{-1}$	0.339	0.226	0.372	0.248	0.407	0.271
$t_r \times 10^3$ (kips)$^{-1}$	0.417	0.278	0.458	0.305	0.501	0.334
r_x/r_y	3.41		3.41		3.41	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W24

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W24×											
		250				229				207			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.454	0.302	0.479	0.319	0.497	0.331	0.528	0.351	0.550	0.366	0.588	0.391
	11	0.517	0.344	0.479	0.319	0.567	0.377	0.528	0.351	0.629	0.419	0.589	0.392
	12	0.530	0.353	0.483	0.322	0.581	0.387	0.534	0.355	0.646	0.430	0.596	0.397
	13	0.544	0.362	0.489	0.325	0.597	0.397	0.540	0.359	0.664	0.442	0.604	0.402
	14	0.560	0.373	0.494	0.329	0.615	0.409	0.547	0.364	0.684	0.455	0.612	0.407
	15	0.578	0.384	0.499	0.332	0.635	0.422	0.553	0.368	0.706	0.470	0.620	0.412
	16	0.597	0.397	0.505	0.336	0.656	0.437	0.560	0.372	0.731	0.486	0.628	0.418
	17	0.619	0.412	0.511	0.340	0.680	0.453	0.567	0.377	0.758	0.505	0.637	0.424
	18	0.642	0.427	0.516	0.344	0.707	0.470	0.574	0.382	0.788	0.525	0.645	0.429
	19	0.668	0.444	0.522	0.347	0.736	0.490	0.581	0.387	0.821	0.547	0.654	0.435
	20	0.696	0.463	0.528	0.351	0.768	0.511	0.588	0.391	0.858	0.571	0.664	0.442
	22	0.762	0.507	0.541	0.360	0.841	0.560	0.604	0.402	0.942	0.626	0.683	0.454
	24	0.840	0.559	0.554	0.369	0.930	0.619	0.620	0.412	1.04	0.694	0.703	0.468
	26	0.935	0.622	0.568	0.378	1.04	0.690	0.637	0.424	1.17	0.775	0.725	0.483
	28	1.05	0.698	0.582	0.387	1.17	0.776	0.655	0.436	1.31	0.874	0.748	0.498
	30	1.19	0.791	0.597	0.397	1.33	0.882	0.674	0.449	1.50	0.996	0.773	0.514
	32	1.35	0.900	0.613	0.408	1.51	1.00	0.694	0.462	1.70	1.13	0.799	0.532
	34	1.53	1.02	0.630	0.419	1.70	1.13	0.716	0.476	1.92	1.28	0.827	0.551
	36	1.71	1.14	0.648	0.431	1.91	1.27	0.739	0.492	2.16	1.43	0.858	0.571
	38	1.91	1.27	0.667	0.444	2.13	1.42	0.763	0.508	2.40	1.60	0.890	0.592
	40	2.11	1.41	0.687	0.457	2.36	1.57	0.789	0.525	2.66	1.77	0.925	0.616
	42	2.33	1.55	0.709	0.472	2.60	1.73	0.817	0.544	2.93	1.95	0.966	0.643
	44	2.56	1.70	0.732	0.487	2.85	1.90	0.847	0.564	3.22	2.14	1.02	0.679
	46	2.80	1.86	0.756	0.503	3.12	2.07	0.884	0.588	3.52	2.34	1.07	0.715
	48	3.05	2.03	0.782	0.520	3.39	2.26	0.928	0.617	3.83	2.55	1.13	0.750
	50	3.30	2.20	0.815	0.542	3.68	2.45	0.971	0.646	4.16	2.77	1.18	0.786

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	2.08	1.39	2.31	1.54	2.60	1.73
$t_y \times 10^3$ (kips)$^{-1}$	0.453	0.302	0.497	0.331	0.549	0.366
$t_r \times 10^3$ (kips)$^{-1}$	0.558	0.372	0.611	0.407	0.677	0.451
r_x/r_y	3.41		3.44		3.44	

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W24

Shape		W24×											
		192				176				162			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.593	0.395	0.637	0.424	0.646	0.430	0.697	0.464	0.700	0.466	0.761	0.506
	11	0.679	0.452	0.639	0.425	0.742	0.494	0.700	0.466	0.802	0.534	0.764	0.508
	12	0.697	0.464	0.647	0.431	0.762	0.507	0.710	0.472	0.824	0.548	0.776	0.516
	13	0.717	0.477	0.656	0.437	0.784	0.521	0.721	0.479	0.847	0.564	0.788	0.524
	14	0.739	0.491	0.665	0.443	0.808	0.538	0.732	0.487	0.874	0.581	0.801	0.533
	15	0.763	0.508	0.675	0.449	0.835	0.556	0.743	0.494	0.903	0.601	0.814	0.541
	16	0.790	0.525	0.685	0.455	0.865	0.576	0.754	0.502	0.935	0.622	0.827	0.550
	17	0.819	0.545	0.695	0.462	0.898	0.598	0.766	0.510	0.971	0.646	0.841	0.560
	18	0.852	0.567	0.705	0.469	0.935	0.622	0.779	0.518	1.01	0.672	0.856	0.569
	19	0.888	0.591	0.715	0.476	0.975	0.649	0.791	0.526	1.05	0.701	0.871	0.579
	20	0.928	0.617	0.726	0.483	1.02	0.678	0.804	0.535	1.10	0.732	0.886	0.589
	22	1.02	0.678	0.749	0.498	1.12	0.746	0.832	0.554	1.21	0.805	0.919	0.611
	24	1.13	0.751	0.773	0.515	1.25	0.829	0.862	0.573	1.34	0.894	0.954	0.635
	26	1.26	0.840	0.799	0.532	1.40	0.929	0.893	0.594	1.50	1.00	0.992	0.660
	28	1.42	0.948	0.827	0.550	1.58	1.05	0.928	0.617	1.70	1.13	1.03	0.687
	30	1.63	1.08	0.857	0.570	1.81	1.20	0.964	0.642	1.94	1.29	1.08	0.717
	32	1.85	1.23	0.889	0.591	2.05	1.37	1.00	0.668	2.21	1.47	1.13	0.749
	34	2.09	1.39	0.923	0.614	2.32	1.54	1.05	0.697	2.49	1.66	1.18	0.784
	36	2.34	1.56	0.961	0.639	2.60	1.73	1.10	0.729	2.80	1.86	1.24	0.827
	38	2.61	1.73	1.00	0.666	2.90	1.93	1.15	0.768	3.12	2.07	1.33	0.887
	40	2.89	1.92	1.05	0.698	3.21	2.14	1.23	0.819	3.45	2.30	1.42	0.948
	42	3.19	2.12	1.11	0.740	3.54	2.35	1.31	0.870	3.81	2.53	1.51	1.01
	44	3.50	2.33	1.18	0.783	3.88	2.58	1.38	0.921	4.18	2.78	1.61	1.07
	46	3.82	2.54	1.24	0.825	4.24	2.82	1.46	0.971	4.57	3.04	1.70	1.13
	48	4.16	2.77	1.30	0.867	4.62	3.07	1.54	1.02	4.97	3.31	1.79	1.19
	50	4.51	3.00	1.37	0.909	5.01	3.34	1.61	1.07	5.39	3.59	1.88	1.25

Other Constants and Properties

	192		176		162	
$b_y \times 10^3$ (kip-ft)$^{-1}$	2.83	1.88	3.10	2.06	3.39	2.26
$t_y \times 10^3$ (kips)$^{-1}$	0.593	0.395	0.645	0.430	0.699	0.466
$t_r \times 10^3$ (kips)$^{-1}$	0.729	0.486	0.794	0.529	0.860	0.573
r_x/r_y	3.42		3.45		3.41	

W24

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W24×											
		146				131				117^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**0.777**	0.517	**0.852**	0.567	**0.867**	0.577	**0.963**	0.641	**0.994**	0.662	**1.09**	0.725
	11	**0.894**	0.595	**0.858**	0.571	**1.00**	0.666	**0.972**	0.646	**1.13**	0.752	**1.10**	0.733
	12	**0.918**	0.611	**0.872**	0.580	**1.03**	0.685	**0.989**	0.658	**1.16**	0.772	**1.12**	0.748
	13	**0.945**	0.629	**0.887**	0.590	**1.06**	0.705	**1.01**	0.670	**1.19**	0.794	**1.15**	0.762
	14	**0.975**	0.649	**0.902**	0.600	**1.09**	0.728	**1.03**	0.683	**1.23**	0.821	**1.17**	0.778
	15	**1.01**	0.671	**0.918**	0.611	**1.13**	0.754	**1.05**	0.696	**1.28**	0.850	**1.19**	0.794
	16	**1.05**	0.696	**0.935**	0.622	**1.18**	0.783	**1.07**	0.710	**1.33**	0.883	**1.22**	0.811
	17	**1.09**	0.723	**0.952**	0.633	**1.22**	0.814	**1.09**	0.724	**1.38**	0.919	**1.24**	0.828
	18	**1.13**	0.753	**0.970**	0.645	**1.28**	0.849	**1.11**	0.739	**1.44**	0.959	**1.27**	0.846
	19	**1.18**	0.786	**0.988**	0.658	**1.33**	0.887	**1.13**	0.754	**1.51**	1.00	**1.30**	0.865
	20	**1.24**	0.823	**1.01**	0.670	**1.40**	0.929	**1.16**	0.770	**1.58**	1.05	**1.33**	0.885
	22	**1.36**	0.907	**1.05**	0.697	**1.54**	1.03	**1.21**	0.804	**1.75**	1.17	**1.39**	0.928
	24	**1.52**	1.01	**1.09**	0.727	**1.72**	1.15	**1.26**	0.841	**1.96**	1.30	**1.46**	0.974
	26	**1.70**	1.13	**1.14**	0.759	**1.94**	1.29	**1.33**	0.882	**2.21**	1.47	**1.54**	1.03
	28	**1.93**	1.29	**1.19**	0.794	**2.21**	1.47	**1.39**	0.928	**2.53**	1.68	**1.63**	1.08
	30	**2.21**	1.47	**1.25**	0.832	**2.54**	1.69	**1.47**	0.978	**2.90**	1.93	**1.73**	1.15
	32	**2.52**	1.68	**1.31**	0.874	**2.89**	1.92	**1.56**	1.04	**3.30**	2.20	**1.89**	1.26
	34	**2.84**	1.89	**1.39**	0.926	**3.26**	2.17	**1.70**	1.13	**3.73**	2.48	**2.07**	1.38
	36	**3.19**	2.12	**1.51**	1.00	**3.65**	2.43	**1.84**	1.23	**4.18**	2.78	**2.25**	1.50
	38	**3.55**	2.36	**1.62**	1.08	**4.07**	2.71	**1.99**	1.32	**4.65**	3.10	**2.43**	1.62
	40	**3.93**	2.62	**1.73**	1.15	**4.51**	3.00	**2.13**	1.42	**5.16**	3.43	**2.62**	1.74
	42	**4.34**	2.89	**1.85**	1.23	**4.97**	3.31	**2.28**	1.52	**5.69**	3.78	**2.80**	1.86
	44	**4.76**	3.17	**1.96**	1.30	**5.46**	3.63	**2.42**	1.61	**6.24**	4.15	**2.99**	1.99
	46	**5.20**	3.46	**2.07**	1.38	**5.96**	3.97	**2.57**	1.71	**6.82**	4.54	**3.17**	2.11
	48	**5.67**	3.77	**2.19**	1.45	**6.49**	4.32	**2.71**	1.80	**7.43**	4.94	**3.35**	2.23
	50	**6.15**	4.09	**2.30**	1.53								

Other Constants and Properties

	146		131		117^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$ $t_y \times 10^3$ (kips)$^{-1}$ $t_r \times 10^3$ (kips)$^{-1}$	**3.82** **0.776** **0.954**	2.54 0.517 0.636	**4.37** **0.866** **1.07**	2.91 0.577 0.710	**4.99** **0.969** **1.19**	3.32 0.646 0.795
r_x/r_y	3.42		3.43		3.44	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W24

Shape	W24×							
	104^c				103^c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending								
0	1.14	0.761	**1.23**	0.820	1.13	0.754	**1.27**	0.847
11	1.30	0.865	**1.25**	0.832	1.52	1.01	**1.42**	0.944
12	1.33	0.886	**1.28**	0.849	1.62	1.08	**1.46**	0.973
13	1.37	0.911	**1.30**	0.867	1.73	1.15	**1.51**	1.00
14	1.41	0.938	**1.33**	0.885	1.86	1.24	**1.56**	1.03
15	1.46	0.969	**1.36**	0.905	2.01	1.34	**1.61**	1.07
16	1.51	1.00	**1.39**	0.925	2.18	1.45	**1.66**	1.11
17	1.57	1.04	**1.42**	0.946	2.38	1.58	**1.72**	1.14
18	1.63	1.09	**1.46**	0.968	2.61	1.74	**1.78**	1.19
19	1.71	1.14	**1.49**	0.991	2.89	1.92	**1.85**	1.23
20	1.79	1.19	**1.53**	1.02	3.20	2.13	**1.92**	1.28
22	1.99	1.32	**1.61**	1.07	3.87	2.57	**2.10**	1.39
24	2.23	1.49	**1.69**	1.13	4.60	3.06	**2.38**	1.58
26	2.53	1.68	**1.79**	1.19	5.40	3.59	**2.66**	1.77
28	2.90	1.93	**1.90**	1.26	6.27	4.17	**2.94**	1.96
30	3.33	2.21	**2.06**	1.37	7.19	4.79	**3.22**	2.15
32	3.78	2.52	**2.28**	1.52	8.18	5.45	**3.51**	2.33
34	4.27	2.84	**2.51**	1.67				
36	4.79	3.19	**2.73**	1.82				
38	5.34	3.55	**2.96**	1.97				
40	5.91	3.93	**3.20**	2.13				
42	6.52	4.34	**3.43**	2.28				
44	7.15	4.76	**3.66**	2.44				
46	7.82	5.20	**3.90**	2.59				
48	8.51	5.66	**4.13**	2.75				

Other Constants and Properties

	104^c ASD	104^c LRFD	103^c ASD	103^c LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	5.71	3.80	8.58	5.71
$t_y \times 10^3$ (kips)$^{-1}$	1.09	0.726	1.10	0.734
$t_r \times 10^3$ (kips)$^{-1}$	1.34	0.893	1.36	0.904
r_x/r_y	3.47		5.03	

^c Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

W24

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

$F_y = 50$ ksi

Shape		W24×											
		94^c				84^c				76			
Design		$p \times 10^3$ $(\text{kips})^{-1}$		$b_x \times 10^3$ $(\text{kip-ft})^{-1}$		$p \times 10^3$ $(\text{kips})^{-1}$		$b_x \times 10^3$ $(\text{kip-ft})^{-1}$		$p \times 10^3$ $(\text{kips})^{-1}$		$b_x \times 10^3$ $(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**1.26**	0.840	**1.40**	0.933	**1.45**	0.968	**1.59**	1.06	**1.64**	1.09	**1.78**	1.19
	6	**1.37**	0.910	**1.40**	0.933	**1.57**	1.05	**1.59**	1.06	**1.78**	1.18	**1.78**	1.19
	7	**1.41**	0.937	**1.40**	0.933	**1.62**	1.08	**1.60**	1.06	**1.83**	1.22	**1.79**	1.19
	8	**1.46**	0.970	**1.44**	0.960	**1.68**	1.12	**1.64**	1.09	**1.90**	1.26	**1.85**	1.23
	9	**1.52**	1.01	**1.48**	0.987	**1.75**	1.16	**1.69**	1.13	**1.97**	1.31	**1.91**	1.27
	10	**1.59**	1.06	**1.53**	1.02	**1.83**	1.22	**1.75**	1.16	**2.06**	1.37	**1.97**	1.31
	11	**1.67**	1.11	**1.58**	1.05	**1.92**	1.28	**1.81**	1.20	**2.17**	1.44	**2.04**	1.36
	12	**1.78**	1.18	**1.63**	1.08	**2.03**	1.35	**1.87**	1.24	**2.30**	1.53	**2.11**	1.41
	13	**1.90**	1.26	**1.68**	1.12	**2.16**	1.44	**1.93**	1.29	**2.45**	1.63	**2.19**	1.46
	14	**2.04**	1.36	**1.74**	1.15	**2.33**	1.55	**2.00**	1.33	**2.63**	1.75	**2.27**	1.51
	15	**2.21**	1.47	**1.80**	1.19	**2.52**	1.68	**2.08**	1.38	**2.84**	1.89	**2.37**	1.57
	16	**2.40**	1.60	**1.86**	1.24	**2.75**	1.83	**2.16**	1.44	**3.10**	2.06	**2.46**	1.64
	17	**2.62**	1.74	**1.93**	1.28	**3.01**	2.00	**2.25**	1.50	**3.41**	2.27	**2.57**	1.71
	18	**2.88**	1.92	**2.01**	1.34	**3.32**	2.21	**2.34**	1.56	**3.77**	2.51	**2.69**	1.79
	19	**3.19**	2.12	**2.09**	1.39	**3.68**	2.45	**2.45**	1.63	**4.19**	2.79	**2.82**	1.87
	20	**3.53**	2.35	**2.18**	1.45	**4.08**	2.71	**2.56**	1.70	**4.65**	3.09	**3.01**	2.00
	22	**4.27**	2.84	**2.43**	1.62	**4.94**	3.28	**2.96**	1.97	**5.62**	3.74	**3.51**	2.33
	24	**5.08**	3.38	**2.77**	1.84	**5.87**	3.91	**3.38**	2.25	**6.69**	4.45	**4.03**	2.68
	26	**5.96**	3.97	**3.11**	2.07	**6.89**	4.59	**3.81**	2.53	**7.85**	5.23	**4.56**	3.03
	28	**6.92**	4.60	**3.45**	2.29	**7.99**	5.32	**4.24**	2.82	**9.11**	6.06	**5.10**	3.39
	30	**7.94**	5.28	**3.79**	2.52	**9.18**	6.11	**4.68**	3.11	**10.5**	6.96	**5.64**	3.75
	32	**9.03**	6.01	**4.14**	2.75	**10.4**	6.95	**5.12**	3.41	**11.9**	7.92	**6.19**	4.12

Other Constants and Properties

	94^c		84^c		76	
$b_y \times 10^3$ $(\text{kip-ft})^{-1}$	**9.50**	6.32	**10.9**	7.27	**12.5**	8.29
$t_y \times 10^3$ $(\text{kips})^{-1}$	**1.20**	0.802	**1.35**	0.899	**1.49**	0.994
$t_r \times 10^3$ $(\text{kips})^{-1}$	**1.48**	0.987	**1.66**	1.11	**1.83**	1.22
r_x/r_y	4.98		5.02		5.05	

^{c} Shape is slender for compression with $F_y = 50$ ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W24

Shape		W24×											
		68^c				62^c				55^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.87	1.25	2.01	1.34	2.07	1.38	2.33	1.55	2.42	1.61	2.66	1.77
	6	2.03	1.35	2.01	1.34	2.40	1.60	2.44	1.63	2.80	1.87	2.82	1.87
	7	2.09	1.39	2.04	1.36	2.54	1.69	2.56	1.70	2.97	1.97	2.96	1.97
	8	2.17	1.44	2.11	1.40	2.72	1.81	2.68	1.78	3.18	2.11	3.11	2.07
	9	2.26	1.50	2.18	1.45	2.94	1.95	2.82	1.87	3.44	2.29	3.28	2.18
	10	2.37	1.57	2.26	1.50	3.22	2.14	2.97	1.97	3.79	2.52	3.47	2.31
	11	2.49	1.66	2.34	1.56	3.58	2.38	3.14	2.09	4.23	2.81	3.68	2.45
	12	2.64	1.76	2.43	1.62	4.06	2.70	3.32	2.21	4.80	3.19	3.92	2.61
	13	2.82	1.88	2.53	1.68	4.67	3.10	3.54	2.35	5.57	3.70	4.19	2.79
	14	3.04	2.02	2.64	1.75	5.41	3.60	3.78	2.51	6.45	4.29	4.54	3.02
	15	3.29	2.19	2.75	1.83	6.21	4.13	4.16	2.77	7.41	4.93	5.12	3.40
	16	3.60	2.39	2.88	1.91	7.07	4.70	4.64	3.08	8.43	5.61	5.71	3.80
	17	3.97	2.64	3.01	2.00	7.98	5.31	5.12	3.41	9.52	6.33	6.33	4.21
	18	4.42	2.94	3.16	2.11	8.94	5.95	5.61	3.73	10.7	7.10	6.96	4.63
	19	4.93	3.28	3.36	2.23	10.0	6.63	6.12	4.07	11.9	7.91	7.60	5.05
	20	5.46	3.63	3.66	2.44	11.0	7.35	6.62	4.41	13.2	8.76	8.25	5.49
	22	6.61	4.40	4.29	2.86	13.4	8.89	7.66	5.10	15.9	10.6	9.58	6.37
	24	7.86	5.23	4.95	3.29								
	26	9.23	6.14	5.62	3.74								
	28	10.7	7.12	6.30	4.19								
	30	12.3	8.18	7.00	4.66								

Other Constants and Properties

	68^c		62^c		55^c	
$b_y \times 10^3$ (kip-ft)	14.5	9.67	22.7	15.1	26.8	17.8
$t_y \times 10^3$ (kips)$^{-1}$	1.66	1.11	1.83	1.22	2.06	1.37
$t_r \times 10^3$ (kips)$^{-1}$	2.04	1.36	2.25	1.50	2.53	1.69
r_x/r_y	5.11		6.69		6.80	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W21

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W21×											
		201				182				166			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.564	0.375	0.672	0.447	0.623	0.414	0.748	0.498	0.684	0.455	0.825	0.549
	6	0.588	0.391	0.672	0.447	0.650	0.432	0.748	0.498	0.714	0.475	0.825	0.549
	7	0.597	0.397	0.672	0.447	0.660	0.439	0.748	0.498	0.725	0.482	0.825	0.549
	8	0.607	0.404	0.672	0.447	0.671	0.447	0.748	0.498	0.738	0.491	0.825	0.549
	9	0.619	0.412	0.672	0.447	0.685	0.456	0.748	0.498	0.753	0.501	0.825	0.549
	10	0.633	0.421	0.672	0.447	0.700	0.466	0.748	0.498	0.770	0.512	0.825	0.549
	11	0.649	0.432	0.675	0.449	0.718	0.478	0.752	0.500	0.789	0.525	0.829	0.552
	12	0.666	0.443	0.682	0.454	0.737	0.491	0.761	0.507	0.811	0.539	0.841	0.559
	13	0.686	0.456	0.690	0.459	0.759	0.505	0.771	0.513	0.835	0.555	0.852	0.567
	14	0.707	0.471	0.698	0.464	0.783	0.521	0.781	0.519	0.862	0.573	0.864	0.575
	15	0.732	0.487	0.706	0.470	0.811	0.539	0.791	0.526	0.892	0.593	0.876	0.583
	16	0.758	0.504	0.714	0.475	0.840	0.559	0.801	0.533	0.925	0.615	0.888	0.591
	17	0.788	0.524	0.723	0.481	0.874	0.581	0.811	0.540	0.962	0.640	0.901	0.599
	18	0.820	0.546	0.731	0.487	0.910	0.605	0.822	0.547	1.00	0.667	0.914	0.608
	19	0.856	0.569	0.740	0.492	0.950	0.632	0.833	0.554	1.05	0.696	0.927	0.617
	20	0.895	0.596	0.749	0.499	0.995	0.662	0.844	0.562	1.10	0.729	0.941	0.626
	22	0.986	0.656	0.768	0.511	1.10	0.730	0.868	0.578	1.21	0.805	0.970	0.646
	24	1.10	0.730	0.788	0.524	1.22	0.813	0.893	0.594	1.35	0.897	1.00	0.666
	26	1.23	0.819	0.809	0.538	1.37	0.914	0.920	0.612	1.52	1.01	1.03	0.688
	28	1.39	0.928	0.831	0.553	1.56	1.04	0.948	0.631	1.72	1.15	1.07	0.712
	30	1.60	1.06	0.854	0.568	1.79	1.19	0.978	0.650	1.98	1.31	1.11	0.737
	32	1.82	1.21	0.879	0.585	2.03	1.35	1.01	0.672	2.25	1.50	1.15	0.764
	34	2.05	1.36	0.905	0.602	2.30	1.53	1.04	0.694	2.54	1.69	1.19	0.793
	36	2.30	1.53	0.933	0.621	2.57	1.71	1.08	0.718	2.84	1.89	1.24	0.824
	38	2.56	1.70	0.962	0.640	2.87	1.91	1.12	0.744	3.17	2.11	1.29	0.858
	40	2.84	1.89	0.994	0.661	3.18	2.11	1.16	0.772	3.51	2.34	1.35	0.897

Other Constants and Properties

	201 ASD	201 LRFD	182 ASD	182 LRFD	166 ASD	166 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	2.68	1.78	2.99	1.99	3.30	2.19
$t_y \times 10^3$ (kips)$^{-1}$	0.563	0.375	0.621	0.414	0.683	0.455
$t_r \times 10^3$ (kips)$^{-1}$	0.693	0.462	0.765	0.510	0.840	0.560
r_x/r_y	3.14		3.13		3.13	

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W21

Shape		W21×											
		147				132				122			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.773**	0.514	**0.955**	0.635	**0.861**	0.573	**1.07**	0.712	**0.931**	0.620	**1.16**	0.772
	6	**0.807**	0.537	**0.955**	0.635	**0.899**	0.598	**1.07**	0.712	**0.974**	0.648	**1.16**	0.772
	7	**0.820**	0.546	**0.955**	0.635	**0.914**	0.608	**1.07**	0.712	**0.990**	0.658	**1.16**	0.772
	8	**0.835**	0.556	**0.955**	0.635	**0.931**	0.619	**1.07**	0.712	**1.01**	0.671	**1.16**	0.772
	9	**0.852**	0.567	**0.955**	0.635	**0.951**	0.632	**1.07**	0.712	**1.03**	0.685	**1.16**	0.772
	10	**0.872**	0.580	**0.955**	0.635	**0.973**	0.647	**1.07**	0.712	**1.05**	0.701	**1.16**	0.772
	11	**0.895**	0.595	**0.963**	0.641	**0.998**	0.664	**1.08**	0.719	**1.08**	0.720	**1.17**	0.781
	12	**0.920**	0.612	**0.978**	0.651	**1.03**	0.683	**1.10**	0.731	**1.11**	0.740	**1.19**	0.795
	13	**0.948**	0.631	**0.993**	0.661	**1.06**	0.704	**1.12**	0.743	**1.15**	0.764	**1.22**	0.809
	14	**0.980**	0.652	**1.01**	0.671	**1.09**	0.728	**1.14**	0.756	**1.19**	0.789	**1.24**	0.823
	15	**1.01**	0.675	**1.02**	0.682	**1.13**	0.755	**1.16**	0.769	**1.23**	0.818	**1.26**	0.838
	16	**1.05**	0.701	**1.04**	0.693	**1.18**	0.784	**1.18**	0.783	**1.28**	0.850	**1.28**	0.854
	17	**1.10**	0.729	**1.06**	0.704	**1.23**	0.816	**1.20**	0.797	**1.33**	0.885	**1.31**	0.870
	18	**1.14**	0.761	**1.08**	0.716	**1.28**	0.852	**1.22**	0.811	**1.39**	0.925	**1.33**	0.887
	19	**1.20**	0.796	**1.09**	0.728	**1.34**	0.892	**1.24**	0.826	**1.45**	0.968	**1.36**	0.905
	20	**1.25**	0.834	**1.11**	0.740	**1.41**	0.935	**1.27**	0.842	**1.53**	1.02	**1.39**	0.923
	22	**1.39**	0.924	**1.15**	0.767	**1.56**	1.04	**1.31**	0.875	**1.69**	1.13	**1.45**	0.962
	24	**1.55**	1.03	**1.19**	0.795	**1.74**	1.16	**1.37**	0.911	**1.90**	1.26	**1.51**	1.00
	26	**1.75**	1.17	**1.24**	0.826	**1.97**	1.31	**1.43**	0.949	**2.15**	1.43	**1.58**	1.05
	28	**2.00**	1.33	**1.29**	0.858	**2.25**	1.50	**1.49**	0.992	**2.46**	1.63	**1.65**	1.10
	30	**2.29**	1.53	**1.34**	0.894	**2.59**	1.72	**1.56**	1.04	**2.82**	1.88	**1.74**	1.16
	32	**2.61**	1.74	**1.40**	0.933	**2.94**	1.96	**1.64**	1.09	**3.21**	2.13	**1.83**	1.22
	34	**2.94**	1.96	**1.47**	0.975	**3.32**	2.21	**1.72**	1.14	**3.62**	2.41	**1.97**	1.31
	36	**3.30**	2.20	**1.54**	1.02	**3.73**	2.48	**1.85**	1.23	**4.06**	2.70	**2.13**	1.41
	38	**3.68**	2.45	**1.64**	1.09	**4.15**	2.76	**1.99**	1.32	**4.52**	3.01	**2.28**	1.52
	40	**4.08**	2.71	**1.75**	1.16	**4.60**	3.06	**2.12**	1.41	**5.01**	3.34	**2.44**	1.62

Other Constants and Properties

	147 ASD	147 LRFD	132 ASD	132 LRFD	122 ASD	122 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	**3.85**	2.56	**4.33**	2.88	**4.71**	3.14
$t_y \times 10^3$ (kips)$^{-1}$	**0.771**	0.514	**0.860**	0.573	**0.930**	0.620
$t_r \times 10^3$ (kips)$^{-1}$	**0.950**	0.633	**1.06**	0.705	**1.14**	0.763
r_x/r_y	3.11		3.11		3.11	

W21

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W21×											
		111				101^c				93			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**1.02**	0.680	**1.28**	0.850	**1.14**	0.755	**1.41**	0.937	**1.22**	0.813	**1.61**	1.07
	6	**1.07**	0.711	**1.28**	0.850	**1.18**	0.786	**1.41**	0.937	**1.37**	0.909	**1.61**	1.07
	7	**1.09**	0.723	**1.28**	0.850	**1.20**	0.798	**1.41**	0.937	**1.42**	0.947	**1.63**	1.09
	8	**1.11**	0.737	**1.28**	0.850	**1.22**	0.811	**1.41**	0.937	**1.49**	0.992	**1.68**	1.12
	9	**1.13**	0.753	**1.28**	0.850	**1.24**	0.827	**1.41**	0.937	**1.57**	1.05	**1.73**	1.15
	10	**1.16**	0.771	**1.28**	0.850	**1.27**	0.847	**1.41**	0.937	**1.67**	1.11	**1.78**	1.18
	11	**1.19**	0.791	**1.29**	0.861	**1.31**	0.870	**1.43**	0.951	**1.78**	1.18	**1.83**	1.22
	12	**1.22**	0.814	**1.32**	0.877	**1.35**	0.895	**1.46**	0.969	**1.91**	1.27	**1.89**	1.25
	13	**1.26**	0.840	**1.34**	0.894	**1.39**	0.924	**1.49**	0.989	**2.07**	1.37	**1.95**	1.29
	14	**1.31**	0.869	**1.37**	0.911	**1.44**	0.956	**1.52**	1.01	**2.25**	1.50	**2.01**	1.34
	15	**1.35**	0.901	**1.40**	0.928	**1.49**	0.992	**1.55**	1.03	**2.46**	1.64	**2.08**	1.38
	16	**1.41**	0.937	**1.42**	0.947	**1.55**	1.03	**1.58**	1.05	**2.71**	1.80	**2.15**	1.43
	17	**1.47**	0.976	**1.45**	0.966	**1.62**	1.07	**1.61**	1.07	**3.00**	2.00	**2.23**	1.48
	18	**1.53**	1.02	**1.48**	0.986	**1.69**	1.12	**1.65**	1.10	**3.35**	2.23	**2.32**	1.54
	19	**1.61**	1.07	**1.51**	1.01	**1.77**	1.18	**1.69**	1.12	**3.74**	2.49	**2.41**	1.60
	20	**1.69**	1.12	**1.55**	1.03	**1.86**	1.24	**1.72**	1.15	**4.14**	2.75	**2.51**	1.67
	22	**1.87**	1.25	**1.62**	1.08	**2.07**	1.37	**1.81**	1.20	**5.01**	3.33	**2.77**	1.84
	24	**2.10**	1.40	**1.69**	1.13	**2.32**	1.54	**1.90**	1.26	**5.96**	3.97	**3.12**	2.07
	26	**2.38**	1.59	**1.78**	1.18	**2.63**	1.75	**2.00**	1.33	**7.00**	4.66	**3.46**	2.30
	28	**2.73**	1.82	**1.87**	1.24	**3.02**	2.01	**2.11**	1.40	**8.12**	5.40	**3.81**	2.54
	30	**3.14**	2.09	**1.97**	1.31	**3.47**	2.31	**2.24**	1.49	**9.32**	6.20	**4.16**	2.77
	32	**3.57**	2.38	**2.12**	1.41	**3.95**	2.63	**2.46**	1.64				
	34	**4.03**	2.68	**2.30**	1.53	**4.46**	2.96	**2.68**	1.79				
	36	**4.52**	3.01	**2.49**	1.66	**5.00**	3.32	**2.91**	1.94				
	38	**5.03**	3.35	**2.68**	1.79	**5.57**	3.70	**3.14**	2.09				
	40	**5.58**	3.71	**2.87**	1.91	**6.17**	4.10	**3.37**	2.24				

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	**5.22**	3.48	**5.77**	3.84	**10.3**	6.83
$t_y \times 10^3$ (kips)$^{-1}$	**1.02**	0.680	**1.12**	0.747	**1.22**	0.813
$t_r \times 10^3$ (kips)$^{-1}$	**1.26**	0.837	**1.38**	0.919	**1.50**	1.00
r_x/r_y	3.12		3.12		4.73	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W21

Shape		W21×											
		83ᶜ				73ᶜ				68ᶜ			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.38	0.919	1.82	1.21	1.62	1.08	2.07	1.38	1.77	1.17	2.23	1.48
	6	1.54	1.02	1.82	1.21	1.78	1.19	2.07	1.38	1.94	1.29	2.23	1.48
	7	1.60	1.06	1.85	1.23	1.85	1.23	2.11	1.40	2.01	1.34	2.27	1.51
	8	1.68	1.12	1.90	1.26	1.93	1.28	2.18	1.45	2.10	1.40	2.35	1.56
	9	1.77	1.18	1.96	1.30	2.02	1.35	2.25	1.50	2.20	1.46	2.43	1.61
	10	1.88	1.25	2.02	1.34	2.14	1.43	2.32	1.55	2.32	1.55	2.51	1.67
	11	2.01	1.34	2.09	1.39	2.29	1.53	2.40	1.60	2.47	1.64	2.60	1.73
	12	2.16	1.44	2.16	1.43	2.47	1.64	2.49	1.66	2.66	1.77	2.70	1.80
	13	2.33	1.55	2.23	1.48	2.68	1.78	2.58	1.72	2.89	1.92	2.81	1.87
	14	2.54	1.69	2.31	1.54	2.92	1.94	2.68	1.79	3.15	2.10	2.93	1.95
	15	2.78	1.85	2.40	1.59	3.20	2.13	2.79	1.86	3.46	2.30	3.05	2.03
	16	3.07	2.04	2.49	1.66	3.54	2.36	2.91	1.94	3.83	2.55	3.19	2.12
	17	3.40	2.26	2.59	1.72	3.94	2.62	3.04	2.02	4.26	2.84	3.34	2.22
	18	3.81	2.53	2.70	1.79	4.41	2.93	3.18	2.11	4.78	3.18	3.50	2.33
	19	4.24	2.82	2.82	1.87	4.91	3.27	3.33	2.22	5.33	3.54	3.72	2.47
	20	4.70	3.13	2.94	1.96	5.45	3.62	3.58	2.38	5.90	3.93	4.03	2.68
	22	5.69	3.78	3.36	2.24	6.59	4.38	4.13	2.75	7.14	4.75	4.66	3.10
	24	6.77	4.50	3.80	2.53	7.84	5.22	4.69	3.12	8.50	5.65	5.30	3.53
	26	7.94	5.29	4.24	2.82	9.20	6.12	5.25	3.49	10.0	6.64	5.95	3.96
	28	9.21	6.13	4.67	3.11	10.7	7.10	5.81	3.87	11.6	7.70	6.60	4.39
	30	10.6	7.04	5.11	3.40	12.3	8.15	6.37	4.24	13.3	8.83	7.25	4.83

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	11.7	7.77	13.4	8.91	14.6	9.71
$t_y \times 10^3$ (kips)$^{-1}$	1.37	0.913	1.55	1.03	1.66	1.11
$t_r \times 10^3$ (kips)$^{-1}$	1.69	1.12	1.91	1.27	2.05	1.37
r_x/r_y	4.74		4.77		4.78	

ᶜ Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W21

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W21×											
		62^c				57^c				55^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**1.98**	1.32	**2.47**	1.65	**2.17**	1.44	**2.76**	1.84	**2.29**	1.52	**2.83**	1.88
	6	**2.18**	1.45	**2.47**	1.65	**2.56**	1.70	**2.91**	1.94	**2.52**	1.68	**2.83**	1.88
	7	**2.26**	1.50	**2.54**	1.69	**2.72**	1.81	**3.04**	2.03	**2.61**	1.74	**2.92**	1.94
	8	**2.36**	1.57	**2.62**	1.74	**2.93**	1.95	**3.19**	2.12	**2.73**	1.81	**3.02**	2.01
	9	**2.47**	1.65	**2.71**	1.81	**3.20**	2.13	**3.35**	2.23	**2.86**	1.91	**3.14**	2.09
	10	**2.61**	1.74	**2.81**	1.87	**3.56**	2.37	**3.53**	2.35	**3.03**	2.01	**3.26**	2.17
	11	**2.78**	1.85	**2.92**	1.94	**4.01**	2.67	**3.73**	2.48	**3.23**	2.15	**3.40**	2.26
	12	**2.98**	1.99	**3.04**	2.02	**4.58**	3.05	**3.95**	2.63	**3.47**	2.31	**3.55**	2.36
	13	**3.23**	2.15	**3.16**	2.10	**5.31**	3.53	**4.20**	2.80	**3.76**	2.50	**3.71**	2.47
	14	**3.54**	2.35	**3.30**	2.19	**6.16**	4.10	**4.49**	2.98	**4.11**	2.73	**3.88**	2.58
	15	**3.90**	2.59	**3.44**	2.29	**7.07**	4.70	**4.94**	3.29	**4.55**	3.03	**4.07**	2.71
	16	**4.33**	2.88	**3.61**	2.40	**8.04**	5.35	**5.47**	3.64	**5.07**	3.37	**4.29**	2.85
	17	**4.84**	3.22	**3.78**	2.52	**9.08**	6.04	**6.01**	4.00	**5.71**	3.80	**4.52**	3.01
	18	**5.43**	3.61	**3.98**	2.65	**10.2**	6.77	**6.55**	4.36	**6.40**	4.26	**4.90**	3.26
	19	**6.05**	4.02	**4.33**	2.88	**11.3**	7.54	**7.10**	4.72	**7.13**	4.74	**5.37**	3.57
	20	**6.70**	4.46	**4.70**	3.13	**12.6**	8.36	**7.65**	5.09	**7.90**	5.26	**5.85**	3.89
	21	**7.39**	4.92	**5.08**	3.38	**13.8**	9.21	**8.20**	5.46	**8.71**	5.80	**6.33**	4.21
	22	**8.11**	5.39	**5.46**	3.63	**15.2**	10.1	**8.76**	5.83	**9.56**	6.36	**6.82**	4.54
	23	**8.86**	5.90	**5.85**	3.89					**10.4**	6.95	**7.32**	4.87
	24	**9.65**	6.42	**6.24**	4.15					**11.4**	7.57	**7.82**	5.20
	25	**10.5**	6.97	**6.63**	4.41					**12.3**	8.21	**8.33**	5.54
	26	**11.3**	7.53	**7.02**	4.67					**13.4**	8.88	**8.84**	5.88
	27	**12.2**	8.13	**7.42**	4.93					**14.4**	9.58	**9.36**	6.23
	28	**13.1**	8.74	**7.81**	5.20					**15.5**	10.3	**9.88**	6.57
	29	**14.1**	9.37	**8.21**	5.46								

Other Constants and Properties

	62^c		57^c		55^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**16.4**	10.9	**24.1**	16.0	**19.4**	12.9
$t_y \times 10^3$ (kips)$^{-1}$	**1.83**	1.22	**1.99**	1.33	**2.06**	1.37
$t_r \times 10^3$ (kips)$^{-1}$	**2.25**	1.50	**2.45**	1.63	**2.53**	1.69
r_x/r_y	4.82		6.18		4.86	

^c Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W21

Shape		W21×											
		50^c				48^{c,f}				44^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	0	2.54	1.69	3.24	2.15	2.70	1.80	3.36	2.23	2.98	1.98	3.73	2.48
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	6	3.01	2.00	3.46	2.30	2.99	1.99	3.36	2.23	3.54	2.36	4.03	2.68
	7	3.21	2.14	3.63	2.41	3.10	2.06	3.47	2.31	3.79	2.52	4.24	2.82
	8	3.47	2.31	3.82	2.54	3.24	2.16	3.61	2.40	4.10	2.73	4.48	2.98
	9	3.81	2.53	4.03	2.68	3.42	2.27	3.76	2.50	4.51	3.00	4.75	3.16
	10	4.24	2.82	4.26	2.84	3.62	2.41	3.92	2.61	5.04	3.35	5.05	3.36
	11	4.83	3.21	4.53	3.01	3.88	2.58	4.10	2.73	5.74	3.82	5.39	3.59
	12	5.57	3.70	4.83	3.21	4.18	2.78	4.30	2.86	6.69	4.45	5.78	3.85
	13	6.51	4.33	5.17	3.44	4.55	3.03	4.51	3.00	7.86	5.23	6.24	4.15
	14	7.55	5.03	5.69	3.79	5.01	3.34	4.74	3.16	9.11	6.06	7.10	4.72
	15	8.67	5.77	6.38	4.24	5.58	3.72	5.00	3.33	10.5	6.96	7.98	5.31
	16	9.87	6.56	7.08	4.71	6.30	4.19	5.30	3.52	11.9	7.92	8.89	5.92
	17	11.1	7.41	7.80	5.19	7.11	4.73	5.74	3.82	13.4	8.94	9.82	6.53
	18	12.5	8.31	8.54	5.68	7.97	5.30	6.34	4.22	15.1	10.0	10.8	7.17
	19	13.9	9.26	9.28	6.17	8.88	5.91	6.96	4.63	16.8	11.2	11.7	7.81
	20	15.4	10.3	10.0	6.67	9.84	6.55	7.59	5.05	18.6	12.4	12.7	8.46
	21	17.0	11.3	10.8	7.17	10.9	7.22	8.24	5.48	20.5	13.6	13.7	9.12
	22					11.9	7.92	8.90	5.92				
	23					13.0	8.66	9.57	6.37				
	24					14.2	9.43	10.3	6.82				
	26					15.4	10.2	10.9	7.28				
	27					16.6	11.1	11.6	7.74				
	28					17.9	11.9	12.3	8.21				
	29												

Other Constants and Properties

	50 ASD	50 LRFD	48 ASD	48 LRFD	44 ASD	44 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	29.2	19.4	24.2	16.1	35.0	23.3
$t_y \times 10^3$ (kips)$^{-1}$	2.27	1.51	2.36	1.57	2.57	1.71
$t_r \times 10^3$ (kips)$^{-1}$	2.79	1.86	2.90	1.94	3.16	2.11
r_x/r_y	6.29		4.96		6.40	

[c] Shape is slender for compression with F_y = 50 ksi.

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W18

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W18×											
		311[h]				283[h]				258[h]			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.365	0.243	0.473	0.314	0.401	0.267	0.527	0.351	0.440	0.293	0.583	0.388
	6	0.381	0.253	0.473	0.314	0.420	0.279	0.527	0.351	0.461	0.307	0.583	0.388
	7	0.387	0.257	0.473	0.314	0.426	0.284	0.527	0.351	0.469	0.312	0.583	0.388
	8	0.394	0.262	0.473	0.314	0.434	0.289	0.527	0.351	0.478	0.318	0.583	0.388
	9	0.402	0.268	0.473	0.314	0.444	0.295	0.527	0.351	0.488	0.325	0.583	0.388
	10	0.412	0.274	0.473	0.314	0.454	0.302	0.527	0.351	0.500	0.333	0.583	0.388
	11	0.422	0.281	0.474	0.315	0.466	0.310	0.530	0.352	0.513	0.342	0.587	0.390
	12	0.434	0.289	0.477	0.317	0.480	0.319	0.533	0.355	0.529	0.352	0.591	0.393
	13	0.447	0.298	0.480	0.319	0.495	0.329	0.537	0.357	0.546	0.363	0.595	0.396
	14	0.462	0.308	0.483	0.321	0.512	0.341	0.540	0.359	0.565	0.376	0.600	0.399
	15	0.479	0.319	0.486	0.323	0.531	0.353	0.544	0.362	0.586	0.390	0.604	0.402
	16	0.497	0.331	0.489	0.325	0.552	0.367	0.547	0.364	0.609	0.405	0.609	0.405
	17	0.517	0.344	0.492	0.327	0.575	0.382	0.551	0.367	0.635	0.423	0.613	0.408
	18	0.540	0.359	0.495	0.329	0.600	0.399	0.555	0.369	0.664	0.442	0.618	0.411
	19	0.564	0.375	0.498	0.331	0.628	0.418	0.559	0.372	0.696	0.463	0.623	0.414
	20	0.592	0.394	0.501	0.333	0.660	0.439	0.563	0.374	0.732	0.487	0.627	0.417
	22	0.655	0.436	0.507	0.338	0.732	0.487	0.571	0.380	0.814	0.541	0.637	0.424
	24	0.732	0.487	0.514	0.342	0.821	0.546	0.579	0.385	0.915	0.609	0.647	0.431
	26	0.826	0.550	0.521	0.346	0.930	0.619	0.587	0.391	1.04	0.691	0.658	0.438
	28	0.942	0.627	0.528	0.351	1.07	0.709	0.596	0.397	1.19	0.794	0.669	0.445
	30	1.08	0.720	0.535	0.356	1.22	0.814	0.605	0.402	1.37	0.912	0.680	0.452
	32	1.23	0.819	0.542	0.361	1.39	0.926	0.614	0.409	1.56	1.04	0.692	0.460
	34	1.39	0.924	0.550	0.366	1.57	1.05	0.624	0.415	1.76	1.17	0.704	0.468
	36	1.56	1.04	0.557	0.371	1.76	1.17	0.633	0.421	1.97	1.31	0.716	0.476
	38	1.74	1.15	0.565	0.376	1.96	1.31	0.644	0.428	2.20	1.46	0.729	0.485
	40	1.92	1.28	0.573	0.381	2.17	1.45	0.654	0.435	2.44	1.62	0.742	0.494

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	1.72	1.15	1.93	1.28	2.15	1.43
$t_y \times 10^3$ (kips)$^{-1}$	0.365	0.243	0.401	0.267	0.440	0.293
$t_r \times 10^3$ (kips)$^{-1}$	0.449	0.299	0.494	0.329	0.542	0.361
r_x/r_y	2.96		2.96		2.96	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W18

Shape	W18×											
	234[h]				211				192			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending: 0	**0.485**	0.323	**0.649**	0.432	**0.537**	0.358	**0.727**	0.484	**0.592**	0.394	**0.806**	0.536
6	**0.508**	0.338	**0.649**	0.432	**0.564**	0.375	**0.727**	0.484	**0.622**	0.414	**0.806**	0.536
7	**0.517**	0.344	**0.649**	0.432	**0.574**	0.382	**0.727**	0.484	**0.633**	0.421	**0.806**	0.536
8	**0.527**	0.351	**0.649**	0.432	**0.585**	0.389	**0.727**	0.484	**0.646**	0.430	**0.806**	0.536
9	**0.539**	0.359	**0.649**	0.432	**0.598**	0.398	**0.727**	0.484	**0.661**	0.440	**0.806**	0.536
10	**0.552**	0.368	**0.649**	0.432	**0.614**	0.408	**0.727**	0.484	**0.678**	0.451	**0.807**	0.537
11	**0.568**	0.378	**0.654**	0.435	**0.631**	0.420	**0.734**	0.488	**0.697**	0.464	**0.815**	0.542
12	**0.585**	0.389	**0.659**	0.438	**0.650**	0.433	**0.740**	0.493	**0.719**	0.479	**0.823**	0.548
13	**0.604**	0.402	**0.664**	0.442	**0.672**	0.447	**0.747**	0.497	**0.744**	0.495	**0.831**	0.553
14	**0.626**	0.416	**0.670**	0.446	**0.697**	0.464	**0.754**	0.502	**0.772**	0.514	**0.840**	0.559
15	**0.650**	0.432	**0.675**	0.449	**0.724**	0.482	**0.761**	0.506	**0.803**	0.534	**0.848**	0.564
16	**0.676**	0.450	**0.681**	0.453	**0.754**	0.502	**0.768**	0.511	**0.837**	0.557	**0.857**	0.570
17	**0.706**	0.470	**0.686**	0.457	**0.788**	0.524	**0.775**	0.516	**0.875**	0.582	**0.866**	0.576
18	**0.738**	0.491	**0.692**	0.461	**0.825**	0.549	**0.782**	0.520	**0.918**	0.611	**0.875**	0.582
19	**0.775**	0.515	**0.698**	0.465	**0.867**	0.577	**0.790**	0.525	**0.965**	0.642	**0.884**	0.588
20	**0.815**	0.542	**0.704**	0.468	**0.913**	0.607	**0.797**	0.530	**1.02**	0.677	**0.894**	0.595
22	**0.909**	0.605	**0.716**	0.477	**1.02**	0.679	**0.813**	0.541	**1.14**	0.758	**0.913**	0.608
24	**1.02**	0.681	**0.729**	0.485	**1.15**	0.767	**0.829**	0.552	**1.29**	0.859	**0.934**	0.621
26	**1.17**	0.775	**0.742**	0.494	**1.32**	0.875	**0.846**	0.563	**1.48**	0.983	**0.955**	0.635
28	**1.34**	0.894	**0.756**	0.503	**1.52**	1.01	**0.864**	0.575	**1.71**	1.14	**0.977**	0.650
30	**1.54**	1.03	**0.770**	0.512	**1.74**	1.16	**0.882**	0.587	**1.96**	1.31	**1.00**	0.666
32	**1.75**	1.17	**0.785**	0.522	**1.99**	1.32	**0.901**	0.600	**2.23**	1.49	**1.03**	0.682
34	**1.98**	1.32	**0.800**	0.532	**2.24**	1.49	**0.921**	0.613	**2.52**	1.68	**1.05**	0.699
36	**2.22**	1.48	**0.816**	0.543	**2.51**	1.67	**0.942**	0.627	**2.83**	1.88	**1.08**	0.718
38	**2.47**	1.65	**0.832**	0.554	**2.80**	1.86	**0.964**	0.641	**3.15**	2.10	**1.11**	0.737
40	**2.74**	1.82	**0.850**	0.565	**3.10**	2.06	**0.987**	0.657	**3.49**	2.32	**1.14**	0.757

Other Constants and Properties

	234[h]		211		192	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**2.39**	1.59	**2.70**	1.80	**2.99**	1.99
$t_y \times 10^3$ (kips)$^{-1}$	**0.485**	0.323	**0.537**	0.358	**0.591**	0.394
$t_r \times 10^3$ (kips)$^{-1}$	**0.596**	0.397	**0.660**	0.440	**0.728**	0.485
r_x/r_y	2.96		2.96		2.97	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W18

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W18×											
		175				158				143			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.651	0.433	0.895	0.596	0.721	0.480	1.00	0.666	0.794	0.528	1.11	0.736
	6	0.684	0.455	0.895	0.596	0.758	0.505	1.00	0.666	0.836	0.556	1.11	0.736
	7	0.696	0.463	0.895	0.596	0.772	0.514	1.00	0.666	0.851	0.567	1.11	0.736
	8	0.711	0.473	0.895	0.596	0.789	0.525	1.00	0.666	0.870	0.579	1.11	0.736
	9	0.728	0.484	0.895	0.596	0.808	0.537	1.00	0.666	0.891	0.593	1.11	0.736
	10	0.747	0.497	0.898	0.597	0.830	0.552	1.00	0.668	0.916	0.609	1.11	0.740
	11	0.769	0.512	0.907	0.604	0.854	0.568	1.02	0.676	0.943	0.628	1.13	0.750
	12	0.794	0.528	0.917	0.610	0.882	0.587	1.03	0.685	0.975	0.649	1.14	0.760
	13	0.822	0.547	0.927	0.617	0.914	0.608	1.04	0.693	1.01	0.672	1.16	0.770
	14	0.853	0.568	0.938	0.624	0.949	0.631	1.05	0.702	1.05	0.698	1.17	0.780
	15	0.888	0.591	0.948	0.631	0.989	0.658	1.07	0.710	1.09	0.728	1.19	0.791
	16	0.927	0.617	0.959	0.638	1.03	0.687	1.08	0.719	1.14	0.761	1.21	0.802
	17	0.970	0.646	0.970	0.646	1.08	0.719	1.10	0.729	1.20	0.797	1.22	0.814
	18	1.02	0.678	0.982	0.653	1.14	0.756	1.11	0.738	1.26	0.838	1.24	0.825
	19	1.07	0.713	0.993	0.661	1.20	0.796	1.12	0.748	1.33	0.883	1.26	0.838
	20	1.13	0.753	1.01	0.669	1.26	0.841	1.14	0.758	1.40	0.934	1.28	0.850
	22	1.27	0.845	1.03	0.685	1.42	0.946	1.17	0.779	1.58	1.05	1.32	0.876
	24	1.44	0.960	1.06	0.702	1.62	1.08	1.20	0.801	1.80	1.20	1.36	0.904
	26	1.66	1.10	1.08	0.721	1.86	1.24	1.24	0.824	2.08	1.38	1.40	0.933
	28	1.92	1.28	1.11	0.740	2.16	1.44	1.28	0.849	2.41	1.61	1.45	0.965
	30	2.21	1.47	1.14	0.760	2.48	1.65	1.32	0.875	2.77	1.84	1.50	0.999
	32	2.51	1.67	1.17	0.781	2.82	1.88	1.36	0.903	3.15	2.10	1.56	1.04
	34	2.83	1.88	1.21	0.804	3.18	2.12	1.40	0.933	3.56	2.37	1.61	1.07
	36	3.18	2.11	1.24	0.827	3.57	2.38	1.45	0.965	3.99	2.65	1.68	1.12
	38	3.54	2.35	1.28	0.853	3.98	2.65	1.50	0.999	4.45	2.96	1.75	1.16
	40	3.92	2.61	1.32	0.880	4.41	2.93	1.56	1.04	4.93	3.28	1.83	1.21

Other Constants and Properties

	175		158		143	
$b_y \times 10^3$ (kip-ft)$^{-1}$	3.36	2.24	3.76	2.50	4.17	2.78
$t_y \times 10^3$ (kips)$^{-1}$	0.650	0.433	0.720	0.480	0.792	0.528
$t_r \times 10^3$ (kips)$^{-1}$	0.800	0.533	0.885	0.590	0.975	0.650
r_x/r_y	2.97		2.96		2.97	

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W18

Shape		W18×											
		130				119				106			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.874**	0.582	**1.23**	0.817	**0.953**	0.634	**1.36**	0.905	**1.07**	0.714	**1.55**	1.03
	6	**0.921**	0.613	**1.23**	0.817	**1.00**	0.668	**1.36**	0.905	**1.13**	0.753	**1.55**	1.03
	7	**0.938**	0.624	**1.23**	0.817	**1.02**	0.681	**1.36**	0.905	**1.15**	0.768	**1.55**	1.03
	8	**0.959**	0.638	**1.23**	0.817	**1.05**	0.696	**1.36**	0.905	**1.18**	0.785	**1.55**	1.03
	9	**0.983**	0.654	**1.23**	0.817	**1.07**	0.713	**1.36**	0.905	**1.21**	0.805	**1.55**	1.03
	10	**1.01**	0.672	**1.24**	0.823	**1.10**	0.733	**1.37**	0.912	**1.24**	0.828	**1.56**	1.04
	11	**1.04**	0.693	**1.25**	0.835	**1.14**	0.756	**1.39**	0.926	**1.28**	0.855	**1.59**	1.06
	12	**1.08**	0.716	**1.27**	0.847	**1.17**	0.782	**1.41**	0.941	**1.33**	0.884	**1.62**	1.08
	13	**1.12**	0.742	**1.29**	0.859	**1.22**	0.811	**1.44**	0.956	**1.38**	0.918	**1.65**	1.10
	14	**1.16**	0.772	**1.31**	0.872	**1.27**	0.843	**1.46**	0.972	**1.44**	0.955	**1.68**	1.12
	15	**1.21**	0.805	**1.33**	0.885	**1.32**	0.879	**1.49**	0.989	**1.50**	0.998	**1.71**	1.14
	16	**1.27**	0.842	**1.35**	0.899	**1.38**	0.920	**1.51**	1.01	**1.57**	1.04	**1.74**	1.16
	17	**1.33**	0.883	**1.37**	0.913	**1.45**	0.965	**1.54**	1.02	**1.65**	1.10	**1.78**	1.18
	18	**1.40**	0.929	**1.39**	0.928	**1.53**	1.02	**1.57**	1.04	**1.74**	1.16	**1.81**	1.21
	19	**1.47**	0.980	**1.42**	0.943	**1.61**	1.07	**1.59**	1.06	**1.84**	1.22	**1.85**	1.23
	20	**1.56**	1.04	**1.44**	0.958	**1.71**	1.13	**1.62**	1.08	**1.95**	1.29	**1.89**	1.26
	22	**1.76**	1.17	**1.49**	0.991	**1.93**	1.28	**1.68**	1.12	**2.20**	1.47	**1.97**	1.31
	24	**2.01**	1.34	**1.54**	1.03	**2.20**	1.47	**1.75**	1.17	**2.53**	1.68	**2.06**	1.37
	26	**2.32**	1.55	**1.60**	1.06	**2.55**	1.70	**1.82**	1.21	**2.94**	1.96	**2.15**	1.43
	28	**2.70**	1.79	**1.66**	1.10	**2.96**	1.97	**1.90**	1.27	**3.41**	2.27	**2.26**	1.50
	30	**3.10**	2.06	**1.72**	1.15	**3.40**	2.26	**1.99**	1.32	**3.91**	2.60	**2.38**	1.58
	32	**3.52**	2.34	**1.79**	1.19	**3.87**	2.57	**2.08**	1.39	**4.45**	2.96	**2.52**	1.67
	34	**3.98**	2.65	**1.87**	1.25	**4.37**	2.90	**2.19**	1.45	**5.03**	3.34	**2.72**	1.81
	36	**4.46**	2.97	**1.96**	1.30	**4.89**	3.26	**2.34**	1.56	**5.64**	3.75	**2.92**	1.94
	38	**4.97**	3.30	**2.07**	1.38	**5.45**	3.63	**2.49**	1.66	**6.28**	4.18	**3.12**	2.08
	40	**5.50**	3.66	**2.20**	1.46	**6.04**	4.02	**2.65**	1.76	**6.96**	4.63	**3.32**	2.21
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		**4.64**		3.09		**5.16**		3.43		**5.89**		3.92	
$t_y \times 10^3$ (kips)$^{-1}$		**0.873**		0.582		**0.951**		0.634		**1.07**		0.714	
$t_r \times 10^3$ (kips)$^{-1}$		**1.07**		0.716		**1.17**		0.780		**1.32**		0.878	
r_x/r_y		2.97				2.94				2.95			

W18

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape	W18×											
	97				86				76^c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending												
0	**1.17**	0.778	**1.69**	1.12	**1.32**	0.878	**1.92**	1.27	**1.52**	1.01	**2.19**	1.45
6	**1.23**	0.822	**1.69**	1.12	**1.39**	0.928	**1.92**	1.27	**1.59**	1.06	**2.19**	1.45
7	**1.26**	0.838	**1.69**	1.12	**1.42**	0.946	**1.92**	1.27	**1.62**	1.08	**2.19**	1.45
8	**1.29**	0.857	**1.69**	1.12	**1.45**	0.968	**1.92**	1.27	**1.66**	1.10	**2.19**	1.45
9	**1.32**	0.879	**1.69**	1.12	**1.49**	0.993	**1.92**	1.27	**1.70**	1.13	**2.19**	1.45
10	**1.36**	0.904	**1.71**	1.14	**1.54**	1.02	**1.94**	1.29	**1.75**	1.16	**2.22**	1.48
11	**1.40**	0.933	**1.74**	1.16	**1.59**	1.06	**1.98**	1.32	**1.81**	1.20	**2.27**	1.51
12	**1.45**	0.966	**1.77**	1.18	**1.64**	1.09	**2.02**	1.35	**1.87**	1.24	**2.32**	1.54
13	**1.51**	1.00	**1.81**	1.20	**1.71**	1.14	**2.06**	1.37	**1.94**	1.29	**2.37**	1.58
14	**1.57**	1.04	**1.84**	1.23	**1.78**	1.18	**2.11**	1.40	**2.03**	1.35	**2.43**	1.62
15	**1.64**	1.09	**1.88**	1.25	**1.86**	1.24	**2.16**	1.43	**2.12**	1.41	**2.49**	1.65
16	**1.72**	1.14	**1.92**	1.28	**1.95**	1.30	**2.20**	1.47	**2.22**	1.48	**2.55**	1.69
17	**1.80**	1.20	**1.96**	1.30	**2.05**	1.36	**2.25**	1.50	**2.34**	1.56	**2.61**	1.74
18	**1.90**	1.27	**2.00**	1.33	**2.16**	1.44	**2.31**	1.53	**2.47**	1.64	**2.68**	1.78
19	**2.01**	1.34	**2.04**	1.36	**2.29**	1.52	**2.36**	1.57	**2.62**	1.74	**2.75**	1.83
20	**2.13**	1.42	**2.09**	1.39	**2.43**	1.61	**2.42**	1.61	**2.78**	1.85	**2.82**	1.88
22	**2.42**	1.61	**2.18**	1.45	**2.76**	1.83	**2.55**	1.69	**3.16**	2.10	**2.98**	1.98
24	**2.77**	1.85	**2.29**	1.52	**3.17**	2.11	**2.68**	1.79	**3.65**	2.43	**3.16**	2.10
26	**3.23**	2.15	**2.41**	1.60	**3.70**	2.46	**2.84**	1.89	**4.26**	2.84	**3.36**	2.24
28	**3.75**	2.49	**2.54**	1.69	**4.29**	2.86	**3.01**	2.01	**4.94**	3.29	**3.67**	2.44
30	**4.30**	2.86	**2.68**	1.78	**4.93**	3.28	**3.29**	2.19	**5.67**	3.78	**4.06**	2.70
32	**4.89**	3.26	**2.91**	1.94	**5.60**	3.73	**3.60**	2.39	**6.46**	4.30	**4.45**	2.96
34	**5.52**	3.68	**3.15**	2.10	**6.33**	4.21	**3.90**	2.60	**7.29**	4.85	**4.85**	3.22
36	**6.19**	4.12	**3.39**	2.25	**7.09**	4.72	**4.21**	2.80	**8.17**	5.44	**5.24**	3.49
38	**6.90**	4.59	**3.63**	2.41	**7.90**	5.26	**4.52**	3.01	**9.10**	6.06	**5.64**	3.75
40	**7.65**	5.09	**3.87**	2.57	**8.76**	5.83	**4.83**	3.21	**10.1**	6.71	**6.04**	4.02

Other Constants and Properties

	97		86		76^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**6.44**	4.29	**7.36**	4.90	**8.44**	5.62
$t_y \times 10^3$ (kips)$^{-1}$	**1.17**	0.778	**1.32**	0.878	**1.50**	1.00
$t_r \times 10^3$ (kips)$^{-1}$	**1.44**	0.958	**1.62**	1.08	**1.84**	1.23
r_x/r_y	2.95		2.95		2.96	

^{c} Shape is slender for compression with F_y = 50 ksi.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W18

Shape		W18×											
		71				65				60^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.60	1.07	2.44	1.62	1.75	1.16	2.68	1.78	1.94	1.29	2.90	1.93
	6	1.83	1.22	2.44	1.62	2.00	1.33	2.68	1.78	2.17	1.45	2.90	1.93
	7	1.92	1.27	2.51	1.67	2.09	1.39	2.76	1.84	2.27	1.51	3.00	1.99
	8	2.02	1.35	2.59	1.72	2.21	1.47	2.85	1.90	2.40	1.60	3.10	2.06
	9	2.15	1.43	2.67	1.78	2.36	1.57	2.95	1.96	2.56	1.70	3.21	2.13
	10	2.31	1.53	2.76	1.83	2.53	1.68	3.05	2.03	2.75	1.83	3.32	2.21
	11	2.49	1.66	2.85	1.90	2.73	1.82	3.15	2.10	2.97	1.98	3.44	2.29
	12	2.71	1.80	2.95	1.96	2.97	1.98	3.27	2.17	3.24	2.16	3.58	2.38
	13	2.97	1.97	3.05	2.03	3.26	2.17	3.39	2.26	3.56	2.37	3.72	2.48
	14	3.27	2.18	3.17	2.11	3.60	2.40	3.52	2.35	3.94	2.62	3.88	2.58
	15	3.64	2.42	3.29	2.19	4.01	2.67	3.67	2.44	4.38	2.92	4.05	2.69
	16	4.07	2.71	3.42	2.28	4.49	2.99	3.83	2.55	4.93	3.28	4.24	2.82
	17	4.60	3.06	3.57	2.37	5.07	3.38	4.00	2.66	5.56	3.70	4.44	2.95
	18	5.15	3.43	3.72	2.48	5.69	3.78	4.18	2.78	6.24	4.15	4.67	3.10
	19	5.74	3.82	3.90	2.59	6.34	4.22	4.41	2.94	6.95	4.62	5.03	3.35
	20	6.36	4.23	4.12	2.74	7.02	4.67	4.75	3.16	7.70	5.12	5.42	3.60
	22	7.70	5.12	4.69	3.12	8.50	5.65	5.42	3.60	9.32	6.20	6.20	4.13
	24	9.16	6.09	5.26	3.50	10.1	6.73	6.09	4.05	11.1	7.38	6.99	4.65
	26	10.7	7.15	5.82	3.87	11.9	7.90	6.76	4.50	13.0	8.66	7.78	5.18
	28	12.5	8.29	6.39	4.25	13.8	9.16	7.43	4.95	15.1	10.0	8.57	5.70

Other Constants and Properties

	71		65		60^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	14.4	9.60	15.8	10.5	17.3	11.5
$t_y \times 10^3$ (kips)$^{-1}$	1.60	1.07	1.75	1.16	1.89	1.26
$t_r \times 10^3$ (kips)$^{-1}$	1.97	1.31	2.15	1.43	2.33	1.55
r_x/r_y	4.41		4.43		4.45	

[c] Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W18

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape	W18×											
	55c				50c				46c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending												
0	**2.14**	1.43	**3.18**	2.12	**2.43**	1.62	**3.53**	2.35	**2.64**	1.76	**3.93**	2.61
6	**2.41**	1.60	**3.19**	2.12	**2.72**	1.81	**3.55**	2.36	**3.18**	2.12	**4.19**	2.79
7	**2.51**	1.67	**3.30**	2.20	**2.84**	1.89	**3.68**	2.45	**3.41**	2.27	**4.39**	2.92
8	**2.64**	1.76	**3.42**	2.27	**2.99**	1.99	**3.81**	2.54	**3.71**	2.47	**4.61**	3.07
9	**2.80**	1.86	**3.55**	2.36	**3.16**	2.11	**3.96**	2.64	**4.12**	2.74	**4.85**	3.23
10	**3.01**	2.00	**3.68**	2.45	**3.38**	2.25	**4.12**	2.74	**4.65**	3.09	**5.12**	3.41
11	**3.26**	2.17	**3.83**	2.55	**3.64**	2.42	**4.29**	2.86	**5.31**	3.53	**5.43**	3.61
12	**3.55**	2.36	**3.99**	2.65	**3.97**	2.64	**4.48**	2.98	**6.14**	4.08	**5.77**	3.84
13	**3.90**	2.60	**4.16**	2.77	**4.38**	2.91	**4.69**	3.12	**7.19**	4.78	**6.16**	4.10
14	**4.32**	2.88	**4.35**	2.89	**4.86**	3.23	**4.91**	3.27	**8.34**	5.55	**6.70**	4.46
15	**4.82**	3.21	**4.55**	3.03	**5.44**	3.62	**5.16**	3.43	**9.57**	6.37	**7.45**	4.96
16	**5.43**	3.61	**4.78**	3.18	**6.14**	4.09	**5.43**	3.62	**10.9**	7.24	**8.21**	5.46
17	**6.13**	4.08	**5.03**	3.35	**6.93**	4.61	**5.76**	3.83	**12.3**	8.18	**8.98**	5.98
18	**6.87**	4.57	**5.39**	3.59	**7.77**	5.17	**6.30**	4.19	**13.8**	9.17	**9.76**	6.49
19	**7.66**	5.09	**5.86**	3.90	**8.66**	5.76	**6.86**	4.56	**15.4**	10.2	**10.5**	7.01
20	**8.48**	5.64	**6.32**	4.21	**9.60**	6.39	**7.42**	4.94	**17.0**	11.3	**11.3**	7.53
21	**9.35**	6.22	**6.79**	4.52	**10.6**	7.04	**7.99**	5.31	**18.8**	12.5	**12.1**	8.05
22	**10.3**	6.83	**7.26**	4.83	**11.6**	7.73	**8.56**	5.69				
23	**11.2**	7.46	**7.73**	5.15	**12.7**	8.45	**9.13**	6.08				
24	**12.2**	8.13	**8.21**	5.46	**13.8**	9.20	**9.71**	6.46				
25	**13.3**	8.82	**8.69**	5.78	**15.0**	10.0	**10.3**	6.85				
26	**14.3**	9.54	**9.16**	6.10	**16.2**	10.8	**10.9**	7.23				
27	**15.5**	10.3	**9.64**	6.41	**17.5**	11.6	**11.5**	7.62				

Other Constants and Properties

	55c		50c		46c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**19.3**	12.8	**21.5**	14.3	**30.5**	20.3
$t_y \times 10^3$ (kips)$^{-1}$	**2.06**	1.37	**2.27**	1.52	**2.46**	1.64
$t_r \times 10^3$ (kips)$^{-1}$	**2.53**	1.69	**2.80**	1.87	**3.03**	2.02
r_x/r_y	4.44		4.47		5.62	

c Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W18

Shape		W18×							
		40^c				35^c			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	3.16	2.10	4.54	3.02	3.71	2.47	5.36	3.56
	6	3.80	2.53	4.88	3.25	4.49	2.99	5.84	3.88
	7	4.08	2.71	5.13	3.41	4.83	3.21	6.16	4.10
	8	4.43	2.95	5.40	3.60	5.28	3.51	6.53	4.34
	9	4.88	3.25	5.71	3.80	5.86	3.90	6.94	4.62
	10	5.47	3.64	6.06	4.03	6.62	4.40	7.41	4.93
	11	6.26	4.16	6.45	4.29	7.64	5.08	7.94	5.28
	12	7.27	4.84	6.89	4.58	9.00	5.99	8.56	5.69
	13	8.53	5.68	7.39	4.92	10.6	7.03	9.59	6.38
	14	9.90	6.59	8.32	5.54	12.3	8.15	10.9	7.23
	15	11.4	7.56	9.29	6.18	14.1	9.36	12.2	8.11
	16	12.9	8.60	10.3	6.84	16.0	10.7	13.5	9.01
	17	14.6	9.71	11.3	7.51	18.1	12.0	14.9	9.92
	18	16.4	10.9	12.3	8.19	20.3	13.5	16.3	10.9
	19	18.2	12.1	13.3	8.87	22.6	15.0	17.7	11.8
	20	20.2	13.4	14.4	9.56	25.0	16.6	19.2	12.7
	21	22.3	14.8	15.4	10.2				

Other Constants and Properties

	40^c ASD	40^c LRFD	35^c ASD	35^c LRFD
$b_y \times 10^3\ (\text{kip-ft})^{-1}$	35.6	23.7	44.2	29.4
$t_y \times 10^3\ (\text{kips})^{-1}$	2.83	1.89	3.24	2.16
$t_r \times 10^3\ (\text{kips})^{-1}$	3.49	2.33	3.99	2.66
r_x/r_y	5.68		5.77	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

W16

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W16×											
		100				89				77			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.13	0.755	1.80	1.20	1.28	0.850	2.04	1.35	1.48	0.983	2.38	1.58
	6	1.20	0.801	1.80	1.20	1.36	0.903	2.04	1.35	1.57	1.05	2.38	1.58
	7	1.23	0.819	1.80	1.20	1.39	0.923	2.04	1.35	1.61	1.07	2.38	1.58
	8	1.26	0.840	1.80	1.20	1.42	0.947	2.04	1.35	1.65	1.10	2.38	1.58
	9	1.30	0.864	1.80	1.20	1.47	0.975	2.04	1.36	1.70	1.13	2.39	1.59
	10	1.34	0.892	1.83	1.22	1.51	1.01	2.08	1.38	1.76	1.17	2.44	1.62
	11	1.39	0.924	1.86	1.24	1.57	1.04	2.12	1.41	1.82	1.21	2.49	1.65
	12	1.44	0.960	1.89	1.26	1.63	1.09	2.16	1.44	1.89	1.26	2.54	1.69
	13	1.50	1.00	1.93	1.28	1.70	1.13	2.20	1.46	1.98	1.32	2.59	1.72
	14	1.57	1.05	1.96	1.30	1.78	1.19	2.24	1.49	2.07	1.38	2.65	1.76
	15	1.65	1.10	1.99	1.33	1.87	1.25	2.29	1.52	2.18	1.45	2.71	1.80
	16	1.74	1.16	2.03	1.35	1.97	1.31	2.33	1.55	2.30	1.53	2.77	1.84
	17	1.84	1.22	2.07	1.38	2.09	1.39	2.38	1.59	2.43	1.62	2.84	1.89
	18	1.95	1.30	2.11	1.40	2.21	1.47	2.43	1.62	2.58	1.72	2.90	1.93
	19	2.07	1.38	2.15	1.43	2.36	1.57	2.49	1.65	2.75	1.83	2.97	1.98
	20	2.21	1.47	2.19	1.46	2.52	1.68	2.54	1.69	2.95	1.96	3.05	2.03
	22	2.55	1.69	2.28	1.52	2.91	1.93	2.66	1.77	3.41	2.27	3.21	2.14
	24	2.97	1.98	2.37	1.58	3.40	2.26	2.79	1.85	4.00	2.66	3.39	2.26
	26	3.49	2.32	2.48	1.65	3.99	2.66	2.93	1.95	4.69	3.12	3.59	2.39
	28	4.05	2.69	2.59	1.72	4.63	3.08	3.09	2.05	5.44	3.62	3.83	2.55
	30	4.65	3.09	2.72	1.81	5.32	3.54	3.26	2.17	6.25	4.16	4.20	2.80
	32	5.29	3.52	2.86	1.90	6.05	4.03	3.53	2.35	7.11	4.73	4.58	3.04
	34	5.97	3.97	3.05	2.03	6.83	4.54	3.81	2.53	8.03	5.34	4.95	3.29
	36	6.69	4.45	3.26	2.17	7.66	5.09	4.08	2.71	9.00	5.99	5.31	3.54
	38	7.46	4.96	3.47	2.31	8.53	5.68	4.35	2.90	10.0	6.67	5.68	3.78
	40	8.26	5.50	3.68	2.45	9.45	6.29	4.62	3.08	11.1	7.39	6.05	4.02

Other Constants and Properties

	W16×100		W16×89		W16×77	
$b_y \times 10^3$ (kip-ft)$^{-1}$	6.49	4.32	7.41	4.93	8.67	5.77
$t_y \times 10^3$ (kips)$^{-1}$	1.13	0.755	1.28	0.850	1.47	0.983
$t_r \times 10^3$ (kips)$^{-1}$	1.39	0.929	1.57	1.05	1.82	1.21
r_x/r_y	2.83		2.83		2.83	

F_y = 50 ksi

Table 6–1 (continued)

Combined Axial and Bending

W Shapes

W16

Shape		W16×											
		67^c				57				50^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.70	1.13	2.74	1.82	1.99	1.33	3.39	2.26	2.29	1.53	3.87	2.58
	6	1.81	1.20	2.74	1.82	2.31	1.54	3.43	2.28	2.63	1.75	3.92	2.61
	7	1.85	1.23	2.74	1.82	2.44	1.62	3.54	2.35	2.78	1.85	4.06	2.70
	8	1.90	1.26	2.74	1.82	2.59	1.72	3.65	2.43	2.96	1.97	4.20	2.80
	9	1.95	1.30	2.76	1.84	2.78	1.85	3.78	2.51	3.17	2.11	4.36	2.90
	10	2.02	1.34	2.82	1.88	3.00	2.00	3.91	2.60	3.43	2.29	4.53	3.01
	11	2.10	1.39	2.88	1.92	3.28	2.18	4.05	2.70	3.75	2.49	4.71	3.13
	12	2.18	1.45	2.95	1.96	3.60	2.40	4.21	2.80	4.13	2.75	4.91	3.27
	13	2.28	1.52	3.02	2.01	3.99	2.66	4.37	2.91	4.58	3.05	5.12	3.41
	14	2.39	1.59	3.09	2.06	4.46	2.97	4.55	3.03	5.12	3.41	5.36	3.56
	15	2.51	1.67	3.17	2.11	5.02	3.34	4.75	3.16	5.78	3.85	5.61	3.74
	16	2.65	1.76	3.25	2.16	5.71	3.80	4.96	3.30	6.58	4.38	5.90	3.92
	17	2.81	1.87	3.33	2.22	6.45	4.29	5.19	3.45	7.43	4.94	6.21	4.13
	18	2.98	1.98	3.42	2.27	7.23	4.81	5.44	3.62	8.33	5.54	6.71	4.47
	19	3.18	2.12	3.51	2.34	8.06	5.36	5.82	3.87	9.28	6.17	7.25	4.83
	20	3.40	2.27	3.61	2.40	8.93	5.94	6.24	4.15	10.3	6.84	7.80	5.19
	22	3.94	2.62	3.82	2.54	10.8	7.19	7.08	4.71	12.4	8.28	8.89	5.92
	24	4.63	3.08	4.07	2.71	12.9	8.55	7.92	5.27	14.8	9.85	9.99	6.65
	26	5.44	3.62	4.34	2.89	15.1	10.0	8.76	5.83	17.4	11.6	11.1	7.37
	28	6.31	4.20	4.82	3.21								
	30	7.24	4.82	5.31	3.53								
	32	8.24	5.48	5.80	3.86								
	34	9.30	6.19	6.29	4.18								
	36	10.4	6.94	6.77	4.51								
	38	11.6	7.73	7.26	4.83								
	40	12.9	8.57	7.75	5.15								

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	10.0	6.68	18.9	12.5	21.9	14.5
$t_y \times 10^3$ (kips)$^{-1}$	1.69	1.13	1.99	1.33	2.26	1.51
$t_r \times 10^3$ (kips)$^{-1}$	2.09	1.39	2.45	1.63	2.78	1.86
r_x/r_y	2.83		4.20		4.20	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W16

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W16×											
		45^c				40^c				36^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	2.61	1.74	4.33	2.88	3.04	2.02	4.88	3.25	3.44	2.29	5.57	3.70
	6	2.98	1.98	4.40	2.93	3.45	2.29	4.96	3.30	3.92	2.61	5.71	3.80
	7	3.13	2.08	4.56	3.03	3.62	2.41	5.16	3.43	4.12	2.74	5.94	3.95
	8	3.31	2.21	4.73	3.15	3.82	2.54	5.36	3.57	4.36	2.90	6.20	4.12
	9	3.56	2.37	4.92	3.27	4.07	2.71	5.59	3.72	4.67	3.10	6.48	4.31
	10	3.86	2.57	5.12	3.41	4.38	2.92	5.83	3.88	5.04	3.35	6.79	4.51
	11	4.22	2.81	5.35	3.56	4.76	3.17	6.10	4.06	5.50	3.66	7.12	4.74
	12	4.66	3.10	5.59	3.72	5.25	3.49	6.40	4.26	6.09	4.05	7.49	4.99
	13	5.18	3.45	5.85	3.89	5.84	3.88	6.72	4.47	6.83	4.54	7.91	5.26
	14	5.82	3.87	6.14	4.08	6.55	4.36	7.08	4.71	7.72	5.14	8.37	5.57
	15	6.59	4.39	6.46	4.30	7.42	4.94	7.48	4.97	8.83	5.87	8.89	5.91
	16	7.50	4.99	6.81	4.53	8.45	5.62	7.97	5.30	10.0	6.68	9.80	6.52
	17	8.47	5.63	7.34	4.88	9.54	6.34	8.77	5.84	11.3	7.54	10.8	7.20
	18	9.49	6.32	8.00	5.32	10.7	7.11	9.59	6.38	12.7	8.46	11.9	7.90
	19	10.6	7.04	8.67	5.77	11.9	7.93	10.4	6.93	14.2	9.42	12.9	8.60
	20	11.7	7.80	9.34	6.22	13.2	8.78	11.3	7.49	15.7	10.4	14.0	9.32
	21	12.9	8.60	10.0	6.67	14.6	9.68	12.1	8.05	17.3	11.5	15.1	10.0
	22	14.2	9.44	10.7	7.12	16.0	10.6	12.9	8.62	19.0	12.6	16.2	10.8
	23	15.5	10.3	11.4	7.57	17.5	11.6	13.8	9.18	20.8	13.8	17.3	11.5
	24	16.9	11.2	12.1	8.02	19.0	12.6	14.7	9.75	22.6	15.0	18.4	12.2
	25	18.3	12.2	12.7	8.48	20.6	13.7	15.5	10.3	24.5	16.3	19.5	13.0
	26	19.8	13.2	13.4	8.93	22.3	14.8	16.4	10.9				

Other Constants and Properties

	45^c ASD	45^c LRFD	40^c ASD	40^c LRFD	36^c ASD	36^c LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	24.6	16.3	28.1	18.7	33.0	21.9
$t_y \times 10^3$ (kips)$^{-1}$	2.51	1.68	2.83	1.89	3.15	2.10
$t_r \times 10^3$ (kips)$^{-1}$	3.09	2.06	3.48	2.32	3.88	2.59
r_x/r_y	4.24		4.22		4.28	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W16

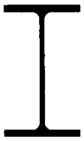
Shape		W16×							
		31^c				26^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	4.09	2.72	6.60	4.39	5.05	3.36	8.06	5.36
	6	5.08	3.38	7.28	4.84	6.33	4.21	9.06	6.03
	7	5.53	3.68	7.71	5.13	6.91	4.59	9.65	6.42
	8	6.11	4.06	8.18	5.45	7.67	5.11	10.3	6.86
	9	6.87	4.57	8.73	5.81	8.70	5.79	11.1	7.37
	10	7.90	5.25	9.35	6.22	10.1	6.71	12.0	7.97
	11	9.28	6.18	10.1	6.70	12.0	8.00	13.0	8.66
	12	11.0	7.35	11.0	7.31	14.3	9.52	15.0	10.0
	13	13.0	8.62	12.5	8.34	16.8	11.2	17.2	11.4
	14	15.0	10.0	14.1	9.40	19.5	13.0	19.4	12.9
	15	17.3	11.5	15.8	10.5	22.4	14.9	21.8	14.5
	16	19.6	13.1	17.4	11.6	25.4	16.9	24.2	16.1
	17	22.2	14.7	19.1	12.7	28.7	19.1	26.6	17.7
	18	24.8	16.5	20.8	13.8	32.2	21.4	29.1	19.3
	19	27.7	18.4	22.5	14.9				

Other Constants and Properties

	W16×31 ASD	W16×31 LRFD	W16×26 ASD	W16×26 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	50.7	33.7	65.0	43.3
$t_y \times 10^3$ (kips)$^{-1}$	3.65	2.44	4.34	2.89
$t_r \times 10^3$ (kips)$^{-1}$	4.50	3.00	5.34	3.56
r_x/r_y	5.48		5.59	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.



W14

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W14×											
		730^h				665^h				605^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.155	0.103	0.215	0.143	0.170	0.113	0.241	0.160	0.188	0.125	0.270	0.180
	11	0.165	0.110	0.215	0.143	0.181	0.120	0.241	0.160	0.200	0.133	0.270	0.180
	12	0.166	0.111	0.215	0.143	0.183	0.122	0.241	0.160	0.202	0.134	0.270	0.180
	13	0.168	0.112	0.215	0.143	0.185	0.123	0.241	0.160	0.204	0.136	0.270	0.180
	14	0.171	0.114	0.215	0.143	0.188	0.125	0.241	0.160	0.207	0.138	0.270	0.180
	15	0.173	0.115	0.215	0.143	0.190	0.127	0.241	0.160	0.210	0.140	0.270	0.180
	16	0.176	0.117	0.215	0.143	0.193	0.129	0.241	0.160	0.214	0.142	0.270	0.180
	17	0.178	0.119	0.215	0.143	0.197	0.131	0.241	0.160	0.217	0.145	0.270	0.180
	18	0.181	0.121	0.215	0.143	0.200	0.133	0.242	0.161	0.221	0.147	0.271	0.180
	19	0.185	0.123	0.216	0.143	0.204	0.135	0.242	0.161	0.225	0.150	0.272	0.181
	20	0.188	0.125	0.216	0.144	0.208	0.138	0.242	0.161	0.230	0.153	0.272	0.181
	22	0.196	0.130	0.217	0.144	0.216	0.144	0.243	0.162	0.240	0.160	0.273	0.182
	24	0.205	0.136	0.217	0.145	0.226	0.151	0.244	0.163	0.252	0.167	0.274	0.183
	26	0.215	0.143	0.218	0.145	0.238	0.158	0.245	0.163	0.265	0.176	0.276	0.183
	28	0.226	0.150	0.219	0.146	0.251	0.167	0.246	0.164	0.280	0.186	0.277	0.184
	30	0.239	0.159	0.220	0.146	0.266	0.177	0.247	0.164	0.297	0.197	0.278	0.185
	32	0.254	0.169	0.221	0.147	0.282	0.188	0.248	0.165	0.316	0.210	0.279	0.186
	34	0.270	0.180	0.221	0.147	0.301	0.201	0.249	0.166	0.338	0.225	0.280	0.187
	36	0.289	0.192	0.222	0.148	0.323	0.215	0.250	0.166	0.363	0.241	0.282	0.187
	38	0.310	0.206	0.223	0.148	0.347	0.231	0.251	0.167	0.391	0.260	0.283	0.188
	40	0.334	0.222	0.224	0.149	0.375	0.250	0.252	0.168	0.423	0.282	0.284	0.189
	42	0.361	0.240	0.225	0.150	0.407	0.271	0.253	0.168	0.460	0.306	0.285	0.190
	44	0.392	0.261	0.226	0.150	0.443	0.295	0.254	0.169	0.503	0.335	0.287	0.191
	46	0.429	0.285	0.226	0.151	0.485	0.322	0.255	0.170	0.550	0.366	0.288	0.191
	48	0.467	0.311	0.227	0.151	0.528	0.351	0.256	0.171	0.599	0.399	0.289	0.192
	50	0.506	0.337	0.228	0.152	0.573	0.381	0.257	0.171	0.650	0.432	0.290	0.193
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		0.437		0.290		0.488		0.325		0.546		0.364	
$t_y \times 10^3$ (kips)$^{-1}$		0.155		0.103		0.170		0.113		0.188		0.125	
$t_r \times 10^3$ (kips)$^{-1}$		0.191		0.127		0.210		0.140		0.231		0.154	
r_x/r_y		1.74				1.73				1.71			

^{h} Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W14

Shape		W14×											
		550^h				500^h				455^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.206**	0.137	**0.302**	0.201	**0.227**	0.151	**0.339**	0.226	**0.249**	0.166	**0.381**	0.253
	11	**0.220**	0.146	**0.302**	0.201	**0.242**	0.161	**0.339**	0.226	**0.266**	0.177	**0.381**	0.253
	12	**0.222**	0.148	**0.302**	0.201	**0.245**	0.163	**0.339**	0.226	**0.270**	0.179	**0.381**	0.253
	13	**0.225**	0.150	**0.302**	0.201	**0.249**	0.166	**0.339**	0.226	**0.273**	0.182	**0.381**	0.253
	14	**0.228**	0.152	**0.302**	0.201	**0.252**	0.168	**0.339**	0.226	**0.278**	0.185	**0.381**	0.253
	15	**0.232**	0.154	**0.302**	0.201	**0.256**	0.171	**0.339**	0.226	**0.282**	0.188	**0.381**	0.253
	16	**0.236**	0.157	**0.302**	0.201	**0.261**	0.173	**0.340**	0.226	**0.287**	0.191	**0.381**	0.254
	17	**0.240**	0.160	**0.303**	0.201	**0.265**	0.177	**0.340**	0.227	**0.292**	0.194	**0.382**	0.254
	18	**0.244**	0.162	**0.303**	0.202	**0.270**	0.180	**0.341**	0.227	**0.298**	0.198	**0.383**	0.255
	19	**0.249**	0.166	**0.304**	0.202	**0.276**	0.183	**0.342**	0.228	**0.304**	0.202	**0.384**	0.256
	20	**0.254**	0.169	**0.305**	0.203	**0.282**	0.187	**0.343**	0.228	**0.310**	0.207	**0.385**	0.256
	22	**0.265**	0.177	**0.306**	0.204	**0.295**	0.196	**0.345**	0.229	**0.325**	0.216	**0.387**	0.258
	24	**0.279**	0.185	**0.308**	0.205	**0.309**	0.206	**0.346**	0.230	**0.342**	0.227	**0.389**	0.259
	26	**0.293**	0.195	**0.309**	0.206	**0.327**	0.217	**0.348**	0.232	**0.361**	0.240	**0.392**	0.261
	28	**0.310**	0.207	**0.310**	0.207	**0.346**	0.230	**0.350**	0.233	**0.383**	0.255	**0.394**	0.262
	30	**0.330**	0.219	**0.312**	0.208	**0.368**	0.245	**0.352**	0.234	**0.408**	0.272	**0.396**	0.263
	32	**0.352**	0.234	**0.313**	0.209	**0.394**	0.262	**0.353**	0.235	**0.437**	0.291	**0.398**	0.265
	34	**0.377**	0.251	**0.315**	0.209	**0.422**	0.281	**0.355**	0.236	**0.470**	0.313	**0.400**	0.266
	36	**0.406**	0.270	**0.316**	0.210	**0.455**	0.303	**0.357**	0.238	**0.508**	0.338	**0.403**	0.268
	38	**0.438**	0.292	**0.318**	0.211	**0.493**	0.328	**0.359**	0.239	**0.551**	0.366	**0.405**	0.269
	40	**0.475**	0.316	**0.319**	0.213	**0.536**	0.357	**0.361**	0.240	**0.600**	0.399	**0.407**	0.271
	42	**0.518**	0.345	**0.321**	0.214	**0.586**	0.390	**0.363**	0.241	**0.657**	0.437	**0.409**	0.272
	44	**0.568**	0.378	**0.322**	0.215	**0.643**	0.428	**0.365**	0.243	**0.721**	0.480	**0.412**	0.274
	46	**0.621**	0.413	**0.324**	0.216	**0.703**	0.468	**0.367**	0.244	**0.789**	0.525	**0.414**	0.276
	48	**0.676**	0.450	**0.326**	0.217	**0.765**	0.509	**0.369**	0.245	**0.859**	0.571	**0.417**	0.277
	50	**0.733**	0.488	**0.327**	0.218	**0.830**	0.552	**0.370**	0.247	**0.932**	0.620	**0.419**	0.279

Other Constants and Properties

	550^h		500^h		455^h	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**0.611**	0.407	**0.683**	0.454	**0.761**	0.506
$t_y \times 10^3$ (kips)$^{-1}$	**0.206**	0.137	**0.227**	0.151	**0.249**	0.166
$t_r \times 10^3$ (kips)$^{-1}$	**0.254**	0.169	**0.279**	0.186	**0.306**	0.204
r_x/r_y	1.70		1.69		1.67	

^{h} Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W14

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W14×											
		426[h]				398[h]				370[h]			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.267**	0.178	**0.410**	0.273	**0.285**	0.190	**0.445**	0.296	**0.306**	0.204	**0.484**	0.322
	11	**0.286**	0.190	**0.410**	0.273	**0.306**	0.203	**0.445**	0.296	**0.329**	0.219	**0.484**	0.322
	12	**0.290**	0.193	**0.410**	0.273	**0.310**	0.206	**0.445**	0.296	**0.333**	0.222	**0.484**	0.322
	13	**0.294**	0.195	**0.410**	0.273	**0.314**	0.209	**0.445**	0.296	**0.338**	0.225	**0.484**	0.322
	14	**0.298**	0.198	**0.410**	0.273	**0.319**	0.212	**0.445**	0.296	**0.343**	0.228	**0.484**	0.322
	15	**0.303**	0.202	**0.410**	0.273	**0.324**	0.216	**0.445**	0.296	**0.349**	0.232	**0.484**	0.322
	16	**0.308**	0.205	**0.411**	0.273	**0.330**	0.220	**0.446**	0.297	**0.355**	0.236	**0.485**	0.323
	17	**0.314**	0.209	**0.412**	0.274	**0.336**	0.224	**0.447**	0.298	**0.362**	0.241	**0.487**	0.324
	18	**0.320**	0.213	**0.413**	0.275	**0.343**	0.228	**0.448**	0.298	**0.369**	0.246	**0.489**	0.325
	19	**0.327**	0.218	**0.414**	0.276	**0.350**	0.233	**0.450**	0.299	**0.377**	0.251	**0.490**	0.326
	20	**0.334**	0.222	**0.415**	0.276	**0.358**	0.238	**0.451**	0.300	**0.386**	0.257	**0.492**	0.327
	22	**0.350**	0.233	**0.418**	0.278	**0.376**	0.250	**0.454**	0.302	**0.405**	0.270	**0.495**	0.329
	24	**0.369**	0.245	**0.420**	0.280	**0.396**	0.263	**0.457**	0.304	**0.427**	0.284	**0.498**	0.331
	26	**0.390**	0.259	**0.423**	0.281	**0.419**	0.279	**0.460**	0.306	**0.453**	0.301	**0.502**	0.334
	28	**0.414**	0.276	**0.425**	0.283	**0.445**	0.296	**0.462**	0.308	**0.482**	0.321	**0.505**	0.336
	30	**0.442**	0.294	**0.428**	0.285	**0.475**	0.316	**0.465**	0.310	**0.515**	0.343	**0.508**	0.338
	32	**0.474**	0.315	**0.430**	0.286	**0.510**	0.339	**0.468**	0.312	**0.554**	0.368	**0.512**	0.340
	34	**0.510**	0.339	**0.433**	0.288	**0.550**	0.366	**0.471**	0.313	**0.597**	0.397	**0.515**	0.343
	36	**0.551**	0.367	**0.435**	0.290	**0.595**	0.396	**0.474**	0.315	**0.648**	0.431	**0.519**	0.345
	38	**0.599**	0.399	**0.438**	0.291	**0.647**	0.431	**0.477**	0.317	**0.705**	0.469	**0.522**	0.347
	40	**0.654**	0.435	**0.441**	0.293	**0.707**	0.470	**0.480**	0.320	**0.772**	0.514	**0.526**	0.350
	42	**0.718**	0.478	**0.443**	0.295	**0.778**	0.517	**0.483**	0.322	**0.850**	0.566	**0.530**	0.352
	44	**0.788**	0.524	**0.446**	0.297	**0.853**	0.568	**0.487**	0.324	**0.933**	0.621	**0.533**	0.355
	46	**0.861**	0.573	**0.449**	0.299	**0.933**	0.621	**0.490**	0.326	**1.02**	0.679	**0.537**	0.357
	48	**0.938**	0.624	**0.451**	0.300	**1.02**	0.676	**0.493**	0.328	**1.11**	0.739	**0.541**	0.360
	50	**1.02**	0.677	**0.454**	0.302	**1.10**	0.733	**0.496**	0.330	**1.21**	0.802	**0.545**	0.362
Other Constants and Properties													
$b_y \times 10^3$ $(\text{kip-ft})^{-1}$		**0.821**		0.546		**0.886**		0.590		**0.963**		0.641	
$t_y \times 10^3$ $(\text{kips})^{-1}$		**0.267**		0.178		**0.285**		0.190		**0.306**		0.204	
$t_r \times 10^3$ $(\text{kips})^{-1}$		**0.329**		0.219		**0.351**		0.234		**0.377**		0.251	
r_x/r_y		1.67				1.66				1.66			

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W14

Shape		W14×											
		342^h				311^h				283^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.331**	0.220	**0.530**	0.353	**0.365**	0.243	**0.591**	0.393	**0.401**	0.267	**0.657**	0.437
	11	**0.355**	0.236	**0.530**	0.353	**0.393**	0.261	**0.591**	0.393	**0.432**	0.287	**0.657**	0.437
	12	**0.360**	0.239	**0.530**	0.353	**0.398**	0.265	**0.591**	0.393	**0.438**	0.291	**0.657**	0.437
	13	**0.365**	0.243	**0.530**	0.353	**0.404**	0.269	**0.591**	0.393	**0.444**	0.296	**0.657**	0.437
	14	**0.371**	0.247	**0.530**	0.353	**0.411**	0.273	**0.591**	0.393	**0.452**	0.301	**0.657**	0.437
	15	**0.377**	0.251	**0.530**	0.353	**0.418**	0.278	**0.591**	0.393	**0.460**	0.306	**0.658**	0.438
	16	**0.384**	0.256	**0.532**	0.354	**0.426**	0.283	**0.593**	0.395	**0.468**	0.312	**0.661**	0.440
	17	**0.392**	0.261	**0.534**	0.355	**0.434**	0.289	**0.596**	0.396	**0.478**	0.318	**0.663**	0.441
	18	**0.400**	0.266	**0.536**	0.356	**0.443**	0.295	**0.598**	0.398	**0.488**	0.325	**0.666**	0.443
	19	**0.409**	0.272	**0.538**	0.358	**0.453**	0.301	**0.600**	0.399	**0.499**	0.332	**0.669**	0.445
	20	**0.418**	0.278	**0.539**	0.359	**0.464**	0.309	**0.602**	0.401	**0.511**	0.340	**0.672**	0.447
	22	**0.439**	0.292	**0.543**	0.361	**0.488**	0.324	**0.607**	0.404	**0.538**	0.358	**0.677**	0.451
	24	**0.463**	0.308	**0.547**	0.364	**0.515**	0.343	**0.612**	0.407	**0.569**	0.378	**0.683**	0.455
	26	**0.491**	0.327	**0.551**	0.367	**0.547**	0.364	**0.617**	0.410	**0.604**	0.402	**0.689**	0.458
	28	**0.523**	0.348	**0.555**	0.369	**0.583**	0.388	**0.621**	0.413	**0.645**	0.429	**0.695**	0.462
	30	**0.560**	0.373	**0.559**	0.372	**0.625**	0.416	**0.626**	0.417	**0.692**	0.460	**0.701**	0.466
	32	**0.602**	0.401	**0.563**	0.375	**0.673**	0.448	**0.631**	0.420	**0.746**	0.496	**0.707**	0.471
	34	**0.651**	0.433	**0.567**	0.377	**0.728**	0.485	**0.636**	0.423	**0.808**	0.537	**0.714**	0.475
	36	**0.706**	0.470	**0.571**	0.380	**0.792**	0.527	**0.642**	0.427	**0.879**	0.585	**0.720**	0.479
	38	**0.770**	0.513	**0.576**	0.383	**0.865**	0.575	**0.647**	0.430	**0.962**	0.640	**0.726**	0.483
	40	**0.844**	0.562	**0.580**	0.386	**0.950**	0.632	**0.652**	0.434	**1.06**	0.704	**0.733**	0.488
	42	**0.931**	0.619	**0.584**	0.389	**1.05**	0.697	**0.658**	0.438	**1.17**	0.777	**0.740**	0.492
	44	**1.02**	0.680	**0.589**	0.392	**1.15**	0.765	**0.663**	0.441	**1.28**	0.852	**0.747**	0.497
	46	**1.12**	0.743	**0.593**	0.395	**1.26**	0.836	**0.669**	0.445	**1.40**	0.932	**0.754**	0.501
	48	**1.22**	0.809	**0.598**	0.398	**1.37**	0.911	**0.674**	0.449	**1.52**	1.01	**0.761**	0.506
	50	**1.32**	0.878	**0.602**	0.401	**1.48**	0.988	**0.680**	0.453	**1.65**	1.10	**0.768**	0.511

Other Constants and Properties

	342^h		311^h		283^h	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**1.05**	0.701	**1.17**	0.780	**1.30**	0.865
$t_y \times 10^3$ (kips)$^{-1}$	**0.330**	0.220	**0.365**	0.243	**0.401**	0.267
$t_r \times 10^3$ (kips)$^{-1}$	**0.407**	0.271	**0.449**	0.299	**0.494**	0.329
r_x/r_y	1.65		1.64		1.63	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W14

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W14×											
		257				233				211			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.442	0.294	0.732	0.487	0.488	0.325	0.817	0.544	0.539	0.359	0.914	0.608
	11	0.476	0.317	0.732	0.487	0.526	0.350	0.817	0.544	0.582	0.387	0.914	0.608
	12	0.483	0.321	0.732	0.487	0.534	0.355	0.817	0.544	0.591	0.393	0.914	0.608
	13	0.491	0.326	0.732	0.487	0.542	0.361	0.817	0.544	0.600	0.399	0.914	0.608
	14	0.499	0.332	0.732	0.487	0.552	0.367	0.817	0.544	0.611	0.406	0.914	0.608
	15	0.508	0.338	0.733	0.488	0.562	0.374	0.819	0.545	0.622	0.414	0.917	0.610
	16	0.518	0.344	0.736	0.490	0.573	0.381	0.823	0.548	0.634	0.422	0.922	0.613
	17	0.528	0.351	0.740	0.492	0.585	0.389	0.827	0.551	0.648	0.431	0.927	0.617
	18	0.540	0.359	0.743	0.494	0.598	0.398	0.832	0.553	0.662	0.441	0.932	0.620
	19	0.552	0.367	0.746	0.497	0.612	0.407	0.836	0.556	0.678	0.451	0.937	0.624
	20	0.566	0.376	0.750	0.499	0.627	0.417	0.840	0.559	0.695	0.463	0.942	0.627
	22	0.596	0.396	0.757	0.503	0.661	0.440	0.849	0.565	0.733	0.488	0.953	0.634
	24	0.631	0.420	0.764	0.508	0.700	0.466	0.857	0.571	0.777	0.517	0.964	0.641
	26	0.671	0.446	0.771	0.513	0.745	0.496	0.866	0.577	0.829	0.551	0.975	0.649
	28	0.717	0.477	0.778	0.518	0.797	0.530	0.876	0.583	0.887	0.590	0.987	0.657
	30	0.770	0.512	0.786	0.523	0.857	0.570	0.885	0.589	0.955	0.636	0.999	0.664
	32	0.832	0.553	0.794	0.528	0.927	0.617	0.895	0.595	1.03	0.688	1.01	0.672
	34	0.902	0.600	0.801	0.533	1.01	0.670	0.904	0.602	1.12	0.748	1.02	0.681
	36	0.984	0.654	0.809	0.539	1.10	0.731	0.914	0.608	1.23	0.818	1.04	0.689
	38	1.08	0.717	0.818	0.544	1.21	0.802	0.925	0.615	1.35	0.898	1.05	0.698
	40	1.19	0.791	0.826	0.549	1.33	0.886	0.935	0.622	1.49	0.994	1.06	0.707
	42	1.31	0.872	0.834	0.555	1.47	0.977	0.946	0.629	1.65	1.10	1.08	0.716
	44	1.44	0.957	0.843	0.561	1.61	1.07	0.957	0.637	1.81	1.20	1.09	0.725
	46	1.57	1.05	0.852	0.567	1.76	1.17	0.968	0.644	1.98	1.31	1.10	0.735
	48	1.71	1.14	0.861	0.573	1.92	1.28	0.980	0.652	2.15	1.43	1.12	0.744
	50	1.86	1.24	0.870	0.579	2.08	1.38	0.991	0.660	2.33	1.55	1.13	0.755

Other Constants and Properties

	257		233		211	
$b_y \times 10^3$ (kip-ft)$^{-1}$	1.45	0.964	1.61	1.07	1.80	1.20
$t_y \times 10^3$ (kips)$^{-1}$	0.441	0.294	0.488	0.325	0.539	0.359
$t_r \times 10^3$ (kips)$^{-1}$	0.543	0.362	0.600	0.400	0.662	0.441
r_x/r_y	1.62		1.62		1.61	

F_y = 50 ksi

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

W14

Shape		W14×											
		193				176				159			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.588	0.391	1.00	0.668	0.645	0.429	1.11	0.741	0.715	0.475	1.24	0.826
	11	0.636	0.423	1.00	0.668	0.698	0.464	1.11	0.741	0.774	0.515	1.24	0.826
	12	0.645	0.429	1.00	0.668	0.709	0.471	1.11	0.741	0.786	0.523	1.24	0.826
	13	0.656	0.436	1.00	0.668	0.720	0.479	1.11	0.741	0.799	0.531	1.24	0.826
	14	0.667	0.444	1.00	0.668	0.733	0.488	1.11	0.741	0.813	0.541	1.24	0.826
	15	0.680	0.452	1.01	0.670	0.747	0.497	1.12	0.745	0.829	0.551	1.25	0.831
	16	0.693	0.461	1.01	0.675	0.762	0.507	1.13	0.750	0.846	0.563	1.26	0.837
	17	0.708	0.471	1.02	0.679	0.779	0.518	1.13	0.755	0.864	0.575	1.27	0.843
	18	0.724	0.482	1.03	0.683	0.797	0.530	1.14	0.760	0.884	0.588	1.28	0.850
	19	0.742	0.494	1.03	0.687	0.816	0.543	1.15	0.765	0.906	0.603	1.29	0.856
	20	0.761	0.506	1.04	0.691	0.837	0.557	1.16	0.770	0.930	0.619	1.30	0.863
	22	0.803	0.534	1.05	0.700	0.884	0.588	1.17	0.781	0.983	0.654	1.32	0.876
	24	0.852	0.567	1.06	0.709	0.939	0.625	1.19	0.791	1.04	0.695	1.34	0.889
	26	0.908	0.604	1.08	0.718	1.00	0.667	1.21	0.803	1.11	0.742	1.36	0.904
	28	0.973	0.648	1.09	0.727	1.08	0.715	1.22	0.814	1.20	0.796	1.38	0.918
	30	1.05	0.698	1.11	0.736	1.16	0.772	1.24	0.826	1.29	0.860	1.40	0.933
	32	1.14	0.755	1.12	0.746	1.26	0.837	1.26	0.838	1.40	0.933	1.43	0.949
	34	1.24	0.822	1.14	0.756	1.37	0.912	1.28	0.851	1.53	1.02	1.45	0.965
	36	1.35	0.899	1.15	0.767	1.50	0.999	1.30	0.864	1.68	1.12	1.47	0.981
	38	1.49	0.989	1.17	0.777	1.65	1.10	1.32	0.877	1.85	1.23	1.50	0.998
	40	1.65	1.10	1.18	0.788	1.83	1.22	1.34	0.891	2.05	1.36	1.53	1.02
	42	1.82	1.21	1.20	0.799	2.02	1.34	1.36	0.905	2.26	1.50	1.56	1.03
	44	1.99	1.33	1.22	0.811	2.22	1.48	1.38	0.920	2.48	1.65	1.58	1.05
	46	2.18	1.45	1.24	0.823	2.42	1.61	1.41	0.935	2.71	1.80	1.61	1.07
	48	2.37	1.58	1.26	0.835	2.64	1.76	1.43	0.951	2.95	1.96	1.64	1.09
	50	2.57	1.71	1.27	0.848	2.86	1.90	1.45	0.967	3.20	2.13	1.68	1.12

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	1.98	1.32	2.19	1.45	2.44	1.62
$t_y \times 10^3$ (kips)$^{-1}$	0.587	0.391	0.644	0.429	0.713	0.475
$t_r \times 10^3$ (kips)$^{-1}$	0.723	0.482	0.792	0.528	0.878	0.585
r_x/r_y	1.60		1.60		1.60	

W14

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

$F_y = 50$ ksi

Shape		W14×											
		145				132				120			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.783	0.521	1.37	0.912	0.862	0.573	1.52	1.01	0.946	0.629	1.68	1.12
	11	0.848	0.564	1.37	0.912	0.943	0.627	1.52	1.01	1.04	0.689	1.68	1.12
	12	0.861	0.573	1.37	0.912	0.959	0.638	1.52	1.01	1.05	0.701	1.68	1.12
	13	0.876	0.583	1.37	0.912	0.977	0.650	1.52	1.01	1.07	0.715	1.68	1.12
	14	0.892	0.593	1.37	0.912	0.997	0.663	1.53	1.02	1.10	0.729	1.69	1.13
	15	0.909	0.605	1.38	0.918	1.02	0.678	1.55	1.03	1.12	0.745	1.71	1.14
	16	0.928	0.617	1.39	0.926	1.04	0.694	1.56	1.04	1.15	0.763	1.73	1.15
	17	0.949	0.631	1.40	0.933	1.07	0.711	1.57	1.05	1.18	0.782	1.74	1.16
	18	0.971	0.646	1.41	0.941	1.10	0.730	1.59	1.06	1.21	0.803	1.76	1.17
	19	0.995	0.662	1.43	0.949	1.13	0.750	1.60	1.07	1.24	0.826	1.78	1.18
	20	1.02	0.679	1.44	0.956	1.16	0.772	1.62	1.08	1.28	0.850	1.80	1.20
	22	1.08	0.719	1.46	0.972	1.24	0.822	1.65	1.10	1.36	0.906	1.84	1.22
	24	1.15	0.764	1.49	0.989	1.32	0.880	1.68	1.12	1.46	0.971	1.88	1.25
	26	1.23	0.816	1.51	1.01	1.43	0.948	1.71	1.14	1.57	1.05	1.92	1.27
	28	1.32	0.877	1.54	1.02	1.54	1.03	1.75	1.16	1.71	1.14	1.96	1.30
	30	1.42	0.947	1.57	1.04	1.68	1.12	1.78	1.19	1.86	1.24	2.00	1.33
	32	1.55	1.03	1.60	1.06	1.85	1.23	1.82	1.21	2.04	1.36	2.05	1.36
	34	1.69	1.12	1.63	1.08	2.04	1.36	1.86	1.24	2.26	1.50	2.10	1.40
	36	1.85	1.23	1.66	1.10	2.27	1.51	1.90	1.27	2.51	1.67	2.15	1.43
	38	2.05	1.36	1.69	1.12	2.52	1.68	1.94	1.29	2.80	1.86	2.21	1.47
	40	2.27	1.51	1.72	1.15	2.80	1.86	1.99	1.32	3.10	2.06	2.26	1.51
	42	2.50	1.66	1.76	1.17	3.08	2.05	2.04	1.35	3.42	2.28	2.32	1.55
	44	2.74	1.83	1.79	1.19	3.38	2.25	2.08	1.39	3.75	2.50	2.39	1.59
	46	3.00	2.00	1.83	1.22	3.70	2.46	2.14	1.42	4.10	2.73	2.45	1.63
	48	3.27	2.17	1.87	1.24	4.03	2.68	2.19	1.46	4.47	2.97	2.53	1.68
	50	3.54	2.36	1.91	1.27	4.37	2.91	2.25	1.49	4.85	3.23	2.60	1.73

Other Constants and Properties

	145 ASD	145 LRFD	132 ASD	132 LRFD	120 ASD	120 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	2.68	1.78	3.15	2.10	3.49	2.32
$t_y \times 10^3$ (kips)$^{-1}$	0.782	0.521	0.860	0.573	0.944	0.629
$t_r \times 10^3$ (kips)$^{-1}$	0.962	0.641	1.06	0.705	1.16	0.774
r_x/r_y	1.59		1.67		1.67	

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W14

Shape		W14×							
		109				99^f			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.04	0.694	1.86	1.23	1.15	0.763	2.07	1.38
	11	1.14	0.760	1.86	1.23	1.26	0.837	2.07	1.38
	12	1.16	0.773	1.86	1.23	1.28	0.851	2.07	1.38
	13	1.18	0.788	1.86	1.23	1.30	0.868	2.07	1.38
	14	1.21	0.804	1.87	1.25	1.33	0.886	2.08	1.38
	15	1.24	0.822	1.89	1.26	1.36	0.906	2.10	1.40
	16	1.27	0.842	1.91	1.27	1.39	0.928	2.13	1.42
	17	1.30	0.863	1.93	1.29	1.43	0.951	2.15	1.43
	18	1.33	0.886	1.95	1.30	1.47	0.977	2.18	1.45
	19	1.37	0.911	1.98	1.31	1.51	1.01	2.21	1.47
	20	1.41	0.939	2.00	1.33	1.56	1.04	2.23	1.49
	22	1.50	1.00	2.04	1.36	1.66	1.10	2.29	1.52
	24	1.61	1.07	2.09	1.39	1.78	1.18	2.35	1.56
	26	1.74	1.16	2.14	1.43	1.92	1.28	2.41	1.60
	28	1.89	1.26	2.20	1.46	2.09	1.39	2.48	1.65
	30	2.06	1.37	2.25	1.50	2.28	1.52	2.55	1.69
	32	2.26	1.51	2.31	1.54	2.51	1.67	2.62	1.74
	34	2.50	1.66	2.37	1.58	2.78	1.85	2.70	1.80
	36	2.79	1.85	2.44	1.62	3.10	2.06	2.78	1.85
	38	3.10	2.06	2.51	1.67	3.45	2.29	2.87	1.91
	40	3.44	2.29	2.58	1.72	3.82	2.54	2.96	1.97
	42	3.79	2.52	2.66	1.77	4.21	2.80	3.06	2.04
	44	4.16	2.77	2.74	1.82	4.62	3.08	3.17	2.11
	46	4.55	3.03	2.83	1.88	5.05	3.36	3.31	2.20
	48	4.95	3.29	2.92	1.94	5.50	3.66	3.48	2.32
	50	5.37	3.57	3.06	2.03	5.97	3.97	3.66	2.43

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	3.84	2.56	4.29	2.85
$t_y \times 10^3$ (kips)$^{-1}$	1.04	0.694	1.14	0.763
$t_r \times 10^3$ (kips)$^{-1}$	1.28	0.854	1.41	0.939
r_x/r_y	1.67		1.66	

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

W14

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W14×											
		90^f				82				74			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.26	0.840	2.33	1.55	1.39	0.924	2.56	1.71	1.53	1.02	2.83	1.88
	6	1.30	0.863	2.33	1.55	1.48	0.983	2.56	1.71	1.63	1.09	2.83	1.88
	7	1.31	0.872	2.33	1.55	1.51	1.00	2.56	1.71	1.67	1.11	2.83	1.88
	8	1.33	0.882	2.33	1.55	1.55	1.03	2.56	1.71	1.71	1.14	2.83	1.88
	9	1.34	0.894	2.33	1.55	1.60	1.06	2.57	1.71	1.76	1.17	2.84	1.89
	10	1.36	0.907	2.33	1.55	1.65	1.10	2.61	1.74	1.82	1.21	2.89	1.92
	11	1.38	0.921	2.33	1.55	1.71	1.14	2.66	1.77	1.89	1.26	2.94	1.96
	12	1.41	0.938	2.33	1.55	1.78	1.18	2.70	1.80	1.96	1.31	2.99	1.99
	13	1.44	0.956	2.33	1.55	1.85	1.23	2.75	1.83	2.05	1.36	3.05	2.03
	14	1.47	0.976	2.33	1.55	1.94	1.29	2.79	1.86	2.14	1.43	3.10	2.07
	15	1.50	0.998	2.33	1.55	2.04	1.36	2.84	1.89	2.25	1.50	3.16	2.10
	16	1.54	1.02	2.35	1.57	2.15	1.43	2.89	1.92	2.38	1.58	3.22	2.15
	17	1.58	1.05	2.38	1.59	2.28	1.52	2.94	1.96	2.52	1.67	3.29	2.19
	18	1.62	1.08	2.42	1.61	2.42	1.61	3.00	1.99	2.67	1.78	3.35	2.23
	19	1.67	1.11	2.45	1.63	2.58	1.71	3.05	2.03	2.85	1.89	3.42	2.28
	20	1.72	1.14	2.48	1.65	2.75	1.83	3.11	2.07	3.04	2.02	3.50	2.33
	22	1.83	1.22	2.55	1.70	3.18	2.12	3.23	2.15	3.51	2.34	3.65	2.43
	24	1.97	1.31	2.62	1.74	3.73	2.48	3.37	2.24	4.12	2.74	3.82	2.54
	26	2.12	1.41	2.70	1.79	4.38	2.91	3.51	2.34	4.83	3.22	4.00	2.66
	28	2.31	1.53	2.78	1.85	5.08	3.38	3.67	2.44	5.61	3.73	4.20	2.80
	30	2.52	1.68	2.86	1.91	5.83	3.88	3.84	2.55	6.44	4.28	4.42	2.94
	32	2.77	1.85	2.95	1.97	6.63	4.41	4.03	2.68	7.32	4.87	4.73	3.15
	34	3.07	2.04	3.05	2.03	7.49	4.98	4.28	2.85	8.27	5.50	5.09	3.38
	36	3.43	2.28	3.16	2.10	8.39	5.59	4.57	3.04	9.27	6.17	5.44	3.62
	38	3.82	2.54	3.27	2.17	9.35	6.22	4.86	3.24	10.3	6.87	5.80	3.86
	40	4.23	2.81	3.39	2.25	10.4	6.90	5.15	3.43	11.4	7.61	6.15	4.09

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	4.90	3.26	7.95	5.29	8.80	5.85
$t_y \times 10^3$ (kips)$^{-1}$	1.26	0.840	1.39	0.924	1.53	1.02
$t_r \times 10^3$ (kips)$^{-1}$	1.55	1.03	1.71	1.14	1.88	1.26
r_x/r_y	1.66		2.44		2.44	

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W14

Shape		W14×											
		68				61				53			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.67	1.11	3.10	2.06	1.86	1.24	3.49	2.32	2.14	1.42	4.09	2.72
	6	1.78	1.18	3.10	2.06	1.99	1.32	3.49	2.32	2.37	1.58	4.09	2.72
	7	1.82	1.21	3.10	2.06	2.03	1.35	3.49	2.32	2.46	1.64	4.11	2.74
	8	1.87	1.24	3.10	2.06	2.09	1.39	3.49	2.32	2.57	1.71	4.21	2.80
	9	1.92	1.28	3.12	2.07	2.15	1.43	3.52	2.34	2.70	1.80	4.32	2.88
	10	1.99	1.32	3.17	2.11	2.22	1.48	3.59	2.39	2.85	1.90	4.44	2.95
	11	2.06	1.37	3.23	2.15	2.30	1.53	3.66	2.44	3.02	2.01	4.56	3.03
	12	2.15	1.43	3.30	2.19	2.40	1.60	3.74	2.49	3.23	2.15	4.68	3.12
	13	2.24	1.49	3.36	2.24	2.51	1.67	3.82	2.54	3.47	2.31	4.82	3.21
	14	2.35	1.56	3.43	2.28	2.63	1.75	3.90	2.59	3.75	2.49	4.96	3.30
	15	2.47	1.65	3.50	2.33	2.77	1.84	3.99	2.65	4.07	2.71	5.11	3.40
	16	2.61	1.74	3.57	2.38	2.92	1.94	4.08	2.71	4.45	2.96	5.27	3.51
	17	2.76	1.84	3.65	2.43	3.09	2.06	4.17	2.78	4.89	3.25	5.44	3.62
	18	2.94	1.95	3.73	2.48	3.29	2.19	4.27	2.84	5.40	3.59	5.62	3.74
	19	3.13	2.08	3.81	2.53	3.51	2.34	4.38	2.91	6.01	4.00	5.81	3.87
	20	3.35	2.23	3.90	2.59	3.76	2.50	4.49	2.98	6.66	4.43	6.02	4.01
	22	3.88	2.58	4.08	2.72	4.36	2.90	4.72	3.14	8.06	5.36	6.49	4.31
	24	4.56	3.04	4.29	2.85	5.13	3.41	4.98	3.32	9.60	6.38	7.24	4.82
	26	5.36	3.56	4.51	3.00	6.02	4.01	5.28	3.51	11.3	7.49	8.01	5.33
	28	6.21	4.13	4.76	3.17	6.98	4.65	5.66	3.77	13.1	8.69	8.79	5.85
	30	7.13	4.74	5.10	3.39	8.02	5.33	6.20	4.12	15.0	9.97	9.56	6.36
	32	8.11	5.40	5.53	3.68	9.12	6.07	6.73	4.48	17.1	11.3	10.3	6.87
	34	9.16	6.09	5.95	3.96	10.3	6.85	7.27	4.83				
	36	10.3	6.83	6.38	4.24	11.5	7.68	7.80	5.19				
	38	11.4	7.61	6.80	4.52	12.9	8.56	8.33	5.54				
	40	12.7	8.44	7.22	4.81	14.3	9.48	8.86	5.90				

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	9.65	6.42	10.9	7.23	16.2	10.8
$t_y \times 10^3$ (kips)$^{-1}$	1.67	1.11	1.86	1.240	2.14	1.42
$t_r \times 10^3$ (kips)$^{-1}$	2.05	1.37	2.29	1.53	2.63	1.75
r_x/r_y	2.44		2.44		3.07	

Note: Heavy line indicates Kl/r equal to or greater than 200.

W14

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W14×											
		48				43^c				38^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**2.36**	1.57	**4.54**	3.02	**2.67**	1.78	**5.12**	3.41	**3.07**	2.04	**5.79**	3.85
	6	**2.62**	1.74	**4.54**	3.02	**2.94**	1.96	**5.12**	3.41	**3.52**	2.34	**5.90**	3.93
	7	**2.72**	1.81	**4.57**	3.04	**3.06**	2.03	**5.17**	3.44	**3.71**	2.47	**6.12**	4.07
	8	**2.84**	1.89	**4.70**	3.13	**3.20**	2.13	**5.31**	3.54	**3.96**	2.64	**6.36**	4.23
	9	**2.98**	1.99	**4.83**	3.21	**3.36**	2.24	**5.47**	3.64	**4.27**	2.84	**6.61**	4.40
	10	**3.15**	2.10	**4.97**	3.30	**3.55**	2.36	**5.64**	3.75	**4.64**	3.09	**6.89**	4.58
	11	**3.35**	2.23	**5.11**	3.40	**3.78**	2.52	**5.82**	3.87	**5.09**	3.38	**7.19**	4.78
	12	**3.58**	2.38	**5.26**	3.50	**4.05**	2.69	**6.01**	4.00	**5.62**	3.74	**7.52**	5.00
	13	**3.85**	2.56	**5.43**	3.61	**4.36**	2.90	**6.21**	4.13	**6.28**	4.18	**7.87**	5.24
	14	**4.16**	2.77	**5.60**	3.73	**4.72**	3.14	**6.43**	4.27	**7.06**	4.70	**8.27**	5.50
	15	**4.52**	3.01	**5.79**	3.85	**5.14**	3.42	**6.66**	4.43	**8.04**	5.35	**8.71**	5.79
	16	**4.95**	3.29	**5.99**	3.98	**5.63**	3.75	**6.91**	4.60	**9.15**	6.08	**9.19**	6.12
	17	**5.44**	3.62	**6.20**	4.13	**6.20**	4.13	**7.18**	4.77	**10.3**	6.87	**10.0**	6.64
	18	**6.02**	4.00	**6.43**	4.28	**6.89**	4.58	**7.47**	4.97	**11.6**	7.70	**10.9**	7.23
	19	**6.70**	4.46	**6.67**	4.44	**7.67**	5.11	**7.78**	5.18	**12.9**	8.58	**11.7**	7.81
	20	**7.43**	4.94	**6.94**	4.62	**8.50**	5.66	**8.12**	5.40	**14.3**	9.51	**12.6**	8.40
	21	**8.19**	5.45	**7.23**	4.81	**9.37**	6.24	**8.72**	5.80	**15.8**	10.5	**13.5**	9.00
	22	**8.99**	5.98	**7.70**	5.12	**10.3**	6.84	**9.31**	6.20	**17.3**	11.5	**14.4**	9.59
	23	**9.83**	6.54	**8.17**	5.44	**11.2**	7.48	**9.91**	6.59	**18.9**	12.6	**15.3**	10.2
	24	**10.7**	7.12	**8.65**	5.76	**12.2**	8.15	**10.5**	6.99	**20.6**	13.7	**16.2**	10.8
	25	**11.6**	7.72	**9.13**	6.07	**13.3**	8.84	**11.1**	7.39	**22.3**	14.9	**17.1**	11.4
	26	**12.6**	8.35	**9.61**	6.39	**14.4**	9.56	**11.7**	7.78				
	27	**13.5**	9.01	**10.1**	6.71	**15.5**	10.3	**12.3**	8.18				
	28	**14.6**	9.69	**10.6**	7.02	**16.7**	11.1	**12.9**	8.58				
	29	**15.6**	10.4	**11.0**	7.34	**17.9**	11.9	**13.5**	8.98				
	30	**16.7**	11.1	**11.5**	7.65	**19.1**	12.7	**14.1**	9.37				

Other Constants and Properties

	48		43^c		38^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**18.2**	12.1	**20.6**	13.7	**29.4**	19.6
$t_y \times 10^3$ (kips)$^{-1}$	**2.36**	1.57	**2.64**	1.76	**2.99**	1.99
$t_r \times 10^3$ (kips)$^{-1}$	**2.90**	1.93	**3.25**	2.17	**3.68**	2.45
r_x/r_y	3.06		3.08		3.79	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W14

Shape		W14×							
		34^c				30^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	3.50	2.33	6.53	4.34	4.02	2.68	7.53	5.01
	6	4.02	2.67	6.67	4.44	4.64	3.08	7.76	5.16
	7	4.23	2.81	6.94	4.61	4.89	3.25	8.09	5.38
	8	4.49	2.99	7.22	4.80	5.20	3.46	8.44	5.62
	9	4.81	3.20	7.53	5.01	5.59	3.72	8.83	5.88
	10	5.24	3.48	7.87	5.23	6.07	4.04	9.26	6.16
	11	5.76	3.83	8.23	5.48	6.70	4.46	9.73	6.48
	12	6.38	4.25	8.64	5.75	7.47	4.97	10.3	6.82
	13	7.14	4.75	9.08	6.04	8.41	5.60	10.8	7.21
	14	8.07	5.37	9.58	6.37	9.56	6.36	11.5	7.64
	15	9.21	6.13	10.1	6.74	11.0	7.30	12.3	8.20
	16	10.5	6.97	10.9	7.28	12.5	8.31	13.7	9.11
	17	11.8	7.87	12.0	8.00	14.1	9.38	15.1	10.0
	18	13.3	8.82	13.1	8.73	15.8	10.5	16.5	11.0
	19	14.8	9.83	14.2	9.46	17.6	11.7	18.0	11.9
	20	16.4	10.9	15.3	10.2	19.5	13.0	19.4	12.9
	21	18.0	12.0	16.5	10.9	21.5	14.3	20.9	13.9
	22	19.8	13.2	17.6	11.7	23.6	15.7	22.4	14.9
	23	21.6	14.4	18.7	12.4	25.8	17.2	23.9	15.9
	24	23.6	15.7	19.8	13.2	28.1	18.7	25.4	16.9
	25	25.6	17.0	21.0	13.9				

Other Constants and Properties

	34^c ASD	34^c LRFD	30^c ASD	30^c LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	33.6	22.4	39.6	26.4
$t_y \times 10^3$ (kips)$^{-1}$	3.33	2.22	3.77	2.51
$t_r \times 10^3$ (kips)$^{-1}$	4.10	2.74	4.64	3.09
r_x/r_y	3.81		3.85	

[c] Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W14

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape	W14×							
	26^c				22^c			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
0	4.73	3.15	8.86	5.90	5.82	3.87	10.7	7.14
6	6.18	4.11	10.0	6.67	7.65	5.09	12.4	8.24
7	6.85	4.55	10.7	7.09	8.52	5.67	13.3	8.82
8	7.75	5.16	11.4	7.58	9.70	6.45	14.3	9.50
9	9.02	6.00	12.2	8.13	11.3	7.53	15.5	10.3
10	10.7	7.13	13.2	8.77	13.6	9.08	16.8	11.2
11	12.9	8.60	14.3	9.52	16.5	11.0	19.2	12.8
12	15.4	10.2	16.4	10.9	19.6	13.1	22.2	14.8
13	18.0	12.0	18.6	12.4	23.1	15.3	25.3	16.8
14	20.9	13.9	20.8	13.8	26.7	17.8	28.5	18.9
15	24.0	16.0	23.1	15.3	30.7	20.4	31.7	21.1
16	27.3	18.2	25.3	16.9	34.9	23.2	35.0	23.3
17	30.9	20.5	27.6	18.4	39.4	26.2	38.3	25.5
18	34.6	23.0	29.9	19.9				

Other Constants and Properties

	26^c		22^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	64.3	42.8	81.2	54.0
$t_y \times 10^3$ (kips)$^{-1}$	4.33	2.89	5.13	3.42
$t_r \times 10^3$ (kips)$^{-1}$	5.33	3.56	6.32	4.21
r_x/r_y	5.23		5.33	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W12

Shape		W12×											
		336^h				305^h				279^h			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.338**	0.225	**0.591**	0.393	**0.373**	0.248	**0.663**	0.441	**0.408**	0.271	**0.741**	0.493
	6	**0.349**	0.232	**0.591**	0.393	**0.385**	0.256	**0.663**	0.441	**0.422**	0.280	**0.741**	0.493
	7	**0.353**	0.235	**0.591**	0.393	**0.389**	0.259	**0.663**	0.441	**0.427**	0.284	**0.741**	0.493
	8	**0.358**	0.238	**0.591**	0.393	**0.395**	0.263	**0.663**	0.441	**0.433**	0.288	**0.741**	0.493
	9	**0.363**	0.241	**0.591**	0.393	**0.401**	0.267	**0.663**	0.441	**0.439**	0.292	**0.741**	0.493
	10	**0.369**	0.246	**0.591**	0.393	**0.408**	0.271	**0.663**	0.441	**0.447**	0.297	**0.741**	0.493
	11	**0.376**	0.250	**0.591**	0.393	**0.415**	0.276	**0.663**	0.441	**0.456**	0.303	**0.741**	0.493
	12	**0.384**	0.255	**0.591**	0.393	**0.424**	0.282	**0.663**	0.441	**0.466**	0.310	**0.741**	0.493
	13	**0.392**	0.261	**0.592**	0.394	**0.434**	0.289	**0.666**	0.443	**0.476**	0.317	**0.744**	0.495
	14	**0.401**	0.267	**0.594**	0.395	**0.445**	0.296	**0.668**	0.444	**0.488**	0.325	**0.746**	0.497
	15	**0.412**	0.274	**0.596**	0.397	**0.456**	0.304	**0.670**	0.446	**0.502**	0.334	**0.749**	0.499
	16	**0.423**	0.281	**0.598**	0.398	**0.469**	0.312	**0.673**	0.448	**0.516**	0.343	**0.752**	0.500
	17	**0.435**	0.290	**0.600**	0.399	**0.483**	0.322	**0.675**	0.449	**0.532**	0.354	**0.755**	0.502
	18	**0.449**	0.299	**0.602**	0.400	**0.499**	0.332	**0.677**	0.451	**0.550**	0.366	**0.758**	0.504
	19	**0.464**	0.308	**0.604**	0.402	**0.516**	0.343	**0.680**	0.452	**0.569**	0.378	**0.761**	0.506
	20	**0.480**	0.319	**0.606**	0.403	**0.534**	0.355	**0.682**	0.454	**0.590**	0.392	**0.764**	0.508
	22	**0.516**	0.344	**0.610**	0.406	**0.576**	0.383	**0.687**	0.457	**0.637**	0.424	**0.770**	0.512
	24	**0.560**	0.372	**0.614**	0.408	**0.626**	0.416	**0.692**	0.460	**0.693**	0.461	**0.776**	0.516
	26	**0.611**	0.406	**0.618**	0.411	**0.685**	0.456	**0.697**	0.464	**0.760**	0.506	**0.782**	0.520
	28	**0.671**	0.447	**0.622**	0.414	**0.755**	0.502	**0.702**	0.467	**0.840**	0.559	**0.788**	0.525
	30	**0.743**	0.494	**0.626**	0.417	**0.838**	0.557	**0.708**	0.471	**0.935**	0.622	**0.795**	0.529
	32	**0.828**	0.551	**0.631**	0.420	**0.937**	0.623	**0.713**	0.474	**1.05**	0.698	**0.801**	0.533
	34	**0.931**	0.620	**0.635**	0.422	**1.06**	0.703	**0.718**	0.478	**1.18**	0.787	**0.808**	0.537
	36	**1.04**	0.695	**0.639**	0.425	**1.18**	0.788	**0.724**	0.481	**1.33**	0.883	**0.814**	0.542
	38	**1.16**	0.774	**0.644**	0.428	**1.32**	0.878	**0.729**	0.485	**1.48**	0.984	**0.821**	0.546
	40	**1.29**	0.857	**0.648**	0.431	**1.46**	0.973	**0.735**	0.489	**1.64**	1.09	**0.828**	0.551

Other Constants and Properties

	336^h		305^h		279^h	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**1.30**	0.865	**1.46**	0.971	**1.62**	1.08
$t_y \times 10^3$ (kips)$^{-1}$	**0.338**	0.225	**0.372**	0.248	**0.407**	0.271
$t_r \times 10^3$ (kips)$^{-1}$	**0.416**	0.277	**0.458**	0.305	**0.501**	0.334
r_x/r_y	1.85		1.84		1.82	

^h Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

W12

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W12×											
		252^h				230^h				210			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.451	0.300	0.832	0.554	0.493	0.328	0.923	0.614	0.541	0.360	1.02	0.681
	6	0.467	0.310	0.832	0.554	0.511	0.340	0.923	0.614	0.560	0.373	1.02	0.681
	7	0.472	0.314	0.832	0.554	0.517	0.344	0.923	0.614	0.567	0.377	1.02	0.681
	8	0.479	0.319	0.832	0.554	0.525	0.349	0.923	0.614	0.576	0.383	1.02	0.681
	9	0.487	0.324	0.832	0.554	0.533	0.355	0.923	0.614	0.585	0.389	1.02	0.681
	10	0.496	0.330	0.832	0.554	0.543	0.361	0.923	0.614	0.596	0.397	1.02	0.681
	11	0.506	0.336	0.832	0.554	0.554	0.369	0.923	0.614	0.609	0.405	1.02	0.681
	12	0.517	0.344	0.833	0.554	0.566	0.377	0.924	0.615	0.622	0.414	1.03	0.683
	13	0.529	0.352	0.837	0.557	0.580	0.386	0.928	0.618	0.638	0.424	1.03	0.686
	14	0.543	0.361	0.840	0.559	0.596	0.396	0.933	0.621	0.655	0.436	1.04	0.689
	15	0.558	0.371	0.844	0.561	0.612	0.407	0.937	0.623	0.674	0.448	1.04	0.693
	16	0.574	0.382	0.847	0.564	0.631	0.420	0.941	0.626	0.695	0.462	1.05	0.696
	17	0.592	0.394	0.851	0.566	0.651	0.433	0.945	0.629	0.717	0.477	1.05	0.700
	18	0.612	0.407	0.854	0.568	0.673	0.448	0.950	0.632	0.742	0.494	1.06	0.703
	19	0.634	0.422	0.858	0.571	0.698	0.464	0.954	0.635	0.770	0.512	1.06	0.707
	20	0.658	0.438	0.862	0.573	0.724	0.482	0.959	0.638	0.800	0.532	1.07	0.710
	22	0.712	0.474	0.869	0.578	0.785	0.523	0.968	0.644	0.868	0.578	1.08	0.718
	24	0.777	0.517	0.876	0.583	0.858	0.571	0.977	0.650	0.950	0.632	1.09	0.725
	26	0.854	0.568	0.884	0.588	0.945	0.628	0.986	0.656	1.05	0.697	1.10	0.733
	28	0.945	0.629	0.892	0.593	1.05	0.697	0.996	0.663	1.16	0.775	1.11	0.741
	30	1.05	0.702	0.900	0.599	1.17	0.779	1.01	0.669	1.30	0.868	1.12	0.748
	32	1.19	0.790	0.908	0.604	1.32	0.880	1.02	0.676	1.48	0.982	1.14	0.757
	34	1.34	0.892	0.916	0.610	1.49	0.993	1.03	0.682	1.67	1.11	1.15	0.765
	36	1.50	1.00	0.925	0.615	1.67	1.11	1.04	0.689	1.87	1.24	1.16	0.773
	38	1.67	1.11	0.933	0.621	1.86	1.24	1.05	0.696	2.08	1.38	1.18	0.782
	40	1.86	1.23	0.942	0.627	2.07	1.37	1.06	0.703	2.31	1.53	1.19	0.791

Other Constants and Properties

	252^h		230^h		210	
$b_y \times 10^3$ (kip-ft)$^{-1}$	1.82	1.21	2.01	1.34	2.24	1.49
$t_y \times 10^3$ (kips)$^{-1}$	0.450	0.300	0.492	0.328	0.540	0.360
$t_r \times 10^3$ (kips)$^{-1}$	0.554	0.369	0.606	0.404	0.665	0.443
r_x/r_y	1.81		1.80		1.80	

[h] Flange thickness greater than 2 in. Special requirements may apply per AISC Specification Section A3.1c.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W12

Shape		W12×											
		190				170				152			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$		$(\text{kips})^{-1}$		$(\text{kip-ft})^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	0.598	0.398	1.15	0.762	0.668	0.444	1.30	0.862	0.746	0.497	1.47	0.975
	6	0.620	0.413	1.15	0.762	0.693	0.461	1.30	0.862	0.775	0.516	1.47	0.975
	7	0.628	0.418	1.15	0.762	0.702	0.467	1.30	0.862	0.785	0.522	1.47	0.975
	8	0.638	0.424	1.15	0.762	0.713	0.474	1.30	0.862	0.798	0.531	1.47	0.975
	9	0.648	0.431	1.15	0.762	0.725	0.483	1.30	0.862	0.812	0.540	1.47	0.975
	10	0.661	0.440	1.15	0.762	0.739	0.492	1.30	0.862	0.828	0.551	1.47	0.975
	11	0.675	0.449	1.15	0.762	0.755	0.503	1.30	0.862	0.846	0.563	1.47	0.975
	12	0.691	0.459	1.15	0.764	0.773	0.514	1.30	0.865	0.866	0.576	1.47	0.980
	13	0.708	0.471	1.16	0.768	0.793	0.528	1.31	0.870	0.889	0.592	1.48	0.987
	14	0.727	0.484	1.16	0.773	0.815	0.542	1.32	0.876	0.914	0.608	1.49	0.994
	15	0.749	0.498	1.17	0.777	0.839	0.559	1.32	0.881	0.942	0.627	1.50	1.00
	16	0.772	0.514	1.17	0.781	0.866	0.576	1.33	0.887	0.973	0.647	1.51	1.01
	17	0.798	0.531	1.18	0.786	0.896	0.596	1.34	0.892	1.01	0.670	1.52	1.01
	18	0.826	0.550	1.19	0.790	0.928	0.618	1.35	0.898	1.04	0.694	1.54	1.02
	19	0.857	0.570	1.19	0.794	0.964	0.641	1.36	0.903	1.08	0.722	1.55	1.03
	20	0.891	0.593	1.20	0.799	1.00	0.667	1.37	0.909	1.13	0.751	1.56	1.04
	22	0.969	0.645	1.21	0.808	1.09	0.727	1.38	0.921	1.23	0.820	1.58	1.05
	24	1.06	0.707	1.23	0.817	1.20	0.798	1.40	0.932	1.35	0.901	1.60	1.07
	26	1.17	0.781	1.24	0.827	1.33	0.883	1.42	0.945	1.50	1.00	1.63	1.08
	28	1.31	0.870	1.26	0.837	1.48	0.985	1.44	0.957	1.68	1.12	1.65	1.10
	30	1.47	0.976	1.27	0.847	1.67	1.11	1.46	0.970	1.89	1.26	1.68	1.12
	32	1.66	1.11	1.29	0.857	1.89	1.26	1.48	0.983	2.15	1.43	1.70	1.13
	34	1.88	1.25	1.30	0.867	2.14	1.42	1.50	0.997	2.43	1.62	1.73	1.15
	36	2.11	1.40	1.32	0.878	2.40	1.59	1.52	1.01	2.73	1.81	1.76	1.17
	38	2.35	1.56	1.34	0.889	2.67	1.78	1.54	1.03	3.04	2.02	1.79	1.19
	40	2.60	1.73	1.35	0.900	2.96	1.97	1.56	1.04	3.37	2.24	1.82	1.21

Other Constants and Properties

	190		170		152	
$b_y \times 10^3$ (kip-ft)$^{-1}$	2.49	1.66	2.83	1.88	3.21	2.14
$t_y \times 10^3$ (kips)$^{-1}$	0.597	0.398	0.666	0.444	0.746	0.497
$t_r \times 10^3$ (kips)$^{-1}$	0.735	0.490	0.821	0.547	0.917	0.611
r_x/r_y	1.79		1.78		1.77	

W12

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W12×											
		136				120				106			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**0.837**	0.557	**1.66**	1.11	**0.947**	0.630	**1.92**	1.27	**1.07**	0.713	**2.17**	1.45
	6	**0.869**	0.578	**1.66**	1.11	**0.984**	0.655	**1.92**	1.27	**1.11**	0.742	**2.17**	1.45
	7	**0.881**	0.586	**1.66**	1.11	**0.998**	0.664	**1.92**	1.27	**1.13**	0.752	**2.17**	1.45
	8	**0.895**	0.595	**1.66**	1.11	**1.01**	0.675	**1.92**	1.27	**1.15**	0.765	**2.17**	1.45
	9	**0.911**	0.606	**1.66**	1.11	**1.03**	0.687	**1.92**	1.27	**1.17**	0.779	**2.17**	1.45
	10	**0.930**	0.618	**1.66**	1.11	**1.05**	0.701	**1.92**	1.27	**1.20**	0.795	**2.17**	1.45
	11	**0.950**	0.632	**1.66**	1.11	**1.08**	0.717	**1.92**	1.27	**1.22**	0.814	**2.17**	1.45
	12	**0.974**	0.648	**1.68**	1.11	**1.11**	0.735	**1.93**	1.28	**1.25**	0.834	**2.19**	1.46
	13	**1.00**	0.665	**1.69**	1.12	**1.14**	0.755	**1.95**	1.30	**1.29**	0.857	**2.22**	1.47
	14	**1.03**	0.684	**1.70**	1.13	**1.17**	0.778	**1.96**	1.31	**1.33**	0.883	**2.24**	1.49
	15	**1.06**	0.706	**1.71**	1.14	**1.21**	0.802	**1.98**	1.32	**1.37**	0.911	**2.26**	1.50
	16	**1.10**	0.729	**1.73**	1.15	**1.25**	0.829	**2.00**	1.33	**1.42**	0.942	**2.28**	1.52
	17	**1.13**	0.755	**1.74**	1.16	**1.29**	0.859	**2.02**	1.34	**1.47**	0.977	**2.31**	1.53
	18	**1.18**	0.783	**1.75**	1.17	**1.34**	0.892	**2.04**	1.35	**1.53**	1.01	**2.33**	1.55
	19	**1.22**	0.814	**1.77**	1.18	**1.40**	0.928	**2.05**	1.37	**1.59**	1.06	**2.35**	1.57
	20	**1.28**	0.849	**1.78**	1.19	**1.46**	0.968	**2.07**	1.38	**1.66**	1.10	**2.38**	1.58
	22	**1.39**	0.927	**1.81**	1.21	**1.59**	1.06	**2.11**	1.41	**1.82**	1.21	**2.43**	1.62
	24	**1.54**	1.02	**1.84**	1.23	**1.76**	1.17	**2.15**	1.43	**2.01**	1.34	**2.48**	1.65
	26	**1.71**	1.14	**1.87**	1.25	**1.96**	1.30	**2.19**	1.46	**2.24**	1.49	**2.54**	1.69
	28	**1.91**	1.27	**1.91**	1.27	**2.20**	1.46	**2.24**	1.49	**2.52**	1.67	**2.60**	1.73
	30	**2.16**	1.44	**1.94**	1.29	**2.49**	1.66	**2.28**	1.52	**2.86**	1.90	**2.66**	1.77
	32	**2.46**	1.64	**1.97**	1.31	**2.84**	1.89	**2.33**	1.55	**3.26**	2.17	**2.72**	1.81
	34	**2.78**	1.85	**2.01**	1.34	**3.20**	2.13	**2.38**	1.58	**3.67**	2.45	**2.79**	1.86
	36	**3.11**	2.07	**2.05**	1.36	**3.59**	2.39	**2.43**	1.62	**4.12**	2.74	**2.86**	1.90
	38	**3.47**	2.31	**2.09**	1.39	**4.00**	2.66	**2.48**	1.65	**4.59**	3.05	**2.93**	1.95
	40	**3.84**	2.56	**2.13**	1.41	**4.43**	2.95	**2.54**	1.69	**5.09**	3.38	**3.01**	2.00

Other Constants and Properties

	136 ASD	136 LRFD	120 ASD	120 LRFD	106 ASD	106 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	**3.64**	2.42	**4.17**	2.78	**4.74**	3.16
$t_y \times 10^3$ (kips)$^{-1}$	**0.836**	0.557	**0.945**	0.630	**1.07**	0.713
$t_r \times 10^3$ (kips)$^{-1}$	**1.03**	0.685	**1.16**	0.775	**1.32**	0.878
r_x/r_y	1.77		1.76		1.76	

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W12

Shape		W12×											
		96				87				79			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.18	0.788	2.42	1.61	1.31	0.869	2.70	1.80	1.44	0.958	2.99	1.99
	6	1.23	0.820	2.42	1.61	1.36	0.905	2.70	1.80	1.50	0.998	2.99	1.99
	7	1.25	0.832	2.42	1.61	1.38	0.918	2.70	1.80	1.52	1.01	2.99	1.99
	8	1.27	0.846	2.42	1.61	1.40	0.934	2.70	1.80	1.55	1.03	2.99	1.99
	9	1.30	0.862	2.42	1.61	1.43	0.951	2.70	1.80	1.58	1.05	2.99	1.99
	10	1.32	0.880	2.42	1.61	1.46	0.972	2.70	1.80	1.61	1.07	2.99	1.99
	11	1.35	0.901	2.43	1.61	1.50	0.995	2.70	1.80	1.65	1.10	3.00	2.00
	12	1.39	0.924	2.45	1.63	1.53	1.02	2.74	1.82	1.70	1.13	3.04	2.02
	13	1.43	0.950	2.48	1.65	1.58	1.05	2.77	1.84	1.74	1.16	3.08	2.05
	14	1.47	0.978	2.51	1.67	1.63	1.08	2.80	1.86	1.80	1.20	3.12	2.08
	15	1.52	1.01	2.53	1.69	1.68	1.12	2.84	1.89	1.86	1.24	3.16	2.10
	16	1.57	1.05	2.56	1.70	1.74	1.16	2.87	1.91	1.92	1.28	3.21	2.13
	17	1.63	1.08	2.59	1.72	1.80	1.20	2.91	1.93	2.00	1.33	3.25	2.16
	18	1.69	1.13	2.62	1.74	1.88	1.25	2.94	1.96	2.08	1.38	3.30	2.19
	19	1.76	1.17	2.65	1.76	1.96	1.30	2.98	1.98	2.17	1.44	3.34	2.22
	20	1.84	1.23	2.68	1.78	2.04	1.36	3.02	2.01	2.26	1.51	3.39	2.26
	22	2.02	1.34	2.74	1.83	2.24	1.49	3.10	2.06	2.49	1.66	3.49	2.32
	24	2.24	1.49	2.81	1.87	2.49	1.65	3.19	2.12	2.76	1.84	3.60	2.39
	26	2.50	1.66	2.88	1.92	2.78	1.85	3.28	2.18	3.10	2.06	3.71	2.47
	28	2.81	1.87	2.96	1.97	3.14	2.09	3.37	2.24	3.50	2.33	3.83	2.55
	30	3.20	2.13	3.03	2.02	3.58	2.38	3.47	2.31	4.00	2.66	3.96	2.64
	32	3.64	2.42	3.12	2.07	4.07	2.71	3.58	2.38	4.55	3.03	4.10	2.73
	34	4.11	2.74	3.20	2.13	4.60	3.06	3.70	2.46	5.13	3.42	4.25	2.83
	36	4.61	3.07	3.29	2.19	5.15	3.43	3.82	2.54	5.75	3.83	4.41	2.93
	38	5.14	3.42	3.39	2.26	5.74	3.82	3.95	2.63	6.41	4.27	4.58	3.05
	40	5.69	3.79	3.49	2.33	6.36	4.23	4.09	2.72	7.10	4.73	4.77	3.17

Other Constants and Properties

	96 ASD	96 LRFD	87 ASD	87 LRFD	79 ASD	79 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	5.28	3.51	5.90	3.92	6.56	4.37
$t_y \times 10^3$ (kips)$^{-1}$	1.18	0.788	1.30	0.869	1.44	0.958
$t_r \times 10^3$ (kips)$^{-1}$	1.46	0.970	1.61	1.07	1.77	1.18
r_x/r_y	1.76		1.75		1.75	

W12

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W12×											
		72				65^f				58			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	**1.58**	1.05	**3.30**	2.19	**1.75**	1.16	**3.75**	2.50	**1.96**	1.30	**4.12**	2.74
	6	**1.65**	1.10	**3.30**	2.19	**1.82**	1.21	**3.75**	2.50	**2.08**	1.38	**4.12**	2.74
	7	**1.67**	1.11	**3.30**	2.19	**1.85**	1.23	**3.75**	2.50	**2.13**	1.41	**4.12**	2.74
	8	**1.70**	1.13	**3.30**	2.19	**1.88**	1.25	**3.75**	2.50	**2.18**	1.45	**4.12**	2.74
	9	**1.73**	1.15	**3.30**	2.19	**1.92**	1.28	**3.75**	2.50	**2.24**	1.49	**4.13**	2.75
	10	**1.77**	1.18	**3.30**	2.19	**1.96**	1.31	**3.75**	2.50	**2.32**	1.54	**4.21**	2.80
	11	**1.81**	1.21	**3.31**	2.20	**2.01**	1.34	**3.75**	2.50	**2.40**	1.60	**4.28**	2.85
	12	**1.86**	1.24	**3.36**	2.23	**2.07**	1.38	**3.75**	2.50	**2.49**	1.66	**4.36**	2.90
	13	**1.92**	1.27	**3.41**	2.27	**2.13**	1.42	**3.81**	2.54	**2.60**	1.73	**4.44**	2.96
	14	**1.98**	1.31	**3.45**	2.30	**2.19**	1.46	**3.87**	2.58	**2.72**	1.81	**4.53**	3.01
	15	**2.04**	1.36	**3.51**	2.33	**2.27**	1.51	**3.93**	2.62	**2.85**	1.90	**4.62**	3.07
	16	**2.12**	1.41	**3.56**	2.37	**2.35**	1.57	**4.00**	2.66	**3.00**	2.00	**4.71**	3.13
	17	**2.20**	1.46	**3.61**	2.40	**2.44**	1.63	**4.06**	2.70	**3.18**	2.11	**4.81**	3.20
	18	**2.29**	1.52	**3.67**	2.44	**2.54**	1.69	**4.13**	2.75	**3.37**	2.24	**4.91**	3.26
	19	**2.38**	1.59	**3.72**	2.48	**2.66**	1.77	**4.20**	2.80	**3.58**	2.38	**5.01**	3.33
	20	**2.49**	1.66	**3.78**	2.52	**2.78**	1.85	**4.27**	2.84	**3.82**	2.54	**5.12**	3.41
	22	**2.74**	1.83	**3.91**	2.60	**3.06**	2.04	**4.43**	2.95	**4.40**	2.93	**5.35**	3.56
	24	**3.05**	2.03	**4.04**	2.69	**3.40**	2.26	**4.59**	3.06	**5.14**	3.42	**5.60**	3.73
	26	**3.41**	2.27	**4.18**	2.78	**3.82**	2.54	**4.77**	3.17	**6.03**	4.01	**5.88**	3.91
	28	**3.86**	2.57	**4.33**	2.88	**4.33**	2.88	**4.96**	3.30	**6.99**	4.65	**6.19**	4.12
	30	**4.42**	2.94	**4.49**	2.99	**4.95**	3.30	**5.17**	3.44	**8.03**	5.34	**6.55**	4.36
	32	**5.02**	3.34	**4.67**	3.11	**5.64**	3.75	**5.40**	3.59	**9.13**	6.08	**7.09**	4.72
	34	**5.67**	3.77	**4.86**	3.23	**6.36**	4.23	**5.64**	3.75	**10.3**	6.86	**7.63**	5.08
	36	**6.36**	4.23	**5.07**	3.37	**7.13**	4.75	**5.98**	3.98	**11.6**	7.69	**8.18**	5.44
	38	**7.08**	4.71	**5.32**	3.54	**7.95**	5.29	**6.40**	4.26	**12.9**	8.57	**8.71**	5.80
	40	**7.85**	5.22	**5.66**	3.77	**8.81**	5.86	**6.82**	4.54	**14.3**	9.49	**9.25**	6.16
Other Constants and Properties													
$b_y \times 10^3$ (kip-ft)$^{-1}$		**7.24**		4.82		**8.31**		5.53		**11.0**		7.29	
$t_y \times 10^3$ (kips)$^{-1}$		**1.58**		1.05		**1.75**		1.17		**1.95**		1.30	
$t_r \times 10^3$ (kips)$^{-1}$		**1.94**		1.29		**2.15**		1.43		**2.41**		1.60	
r_x/r_y		1.75				1.75				2.10			

^{f} Shape does not meet compact limit for flexure with F_y = 50 ksi.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W12

Shape		W12×											
		53				50				45			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	2.15	1.43	4.57	3.04	2.29	1.52	4.96	3.30	2.55	1.69	5.55	3.69
	6	2.28	1.52	4.57	3.04	2.53	1.68	4.96	3.30	2.81	1.87	5.55	3.69
	7	2.33	1.55	4.57	3.04	2.62	1.74	4.96	3.30	2.92	1.94	5.56	3.70
	8	2.39	1.59	4.57	3.04	2.73	1.81	5.08	3.38	3.04	2.02	5.70	3.79
	9	2.46	1.64	4.59	3.06	2.86	1.90	5.19	3.46	3.19	2.12	5.84	3.89
	10	2.55	1.69	4.68	3.12	3.01	2.00	5.32	3.54	3.36	2.23	6.00	3.99
	11	2.64	1.76	4.77	3.18	3.19	2.12	5.45	3.62	3.56	2.37	6.15	4.09
	12	2.75	1.83	4.87	3.24	3.40	2.26	5.58	3.71	3.79	2.52	6.32	4.21
	13	2.87	1.91	4.97	3.31	3.64	2.42	5.73	3.81	4.06	2.70	6.50	4.32
	14	3.00	2.00	5.07	3.37	3.92	2.61	5.88	3.91	4.38	2.91	6.69	4.45
	15	3.15	2.10	5.18	3.45	4.24	2.82	6.03	4.01	4.75	3.16	6.88	4.58
	16	3.33	2.21	5.29	3.52	4.62	3.07	6.20	4.13	5.17	3.44	7.09	4.72
	17	3.52	2.34	5.41	3.60	5.05	3.36	6.38	4.24	5.67	3.77	7.32	4.87
	18	3.74	2.49	5.53	3.68	5.56	3.70	6.56	4.37	6.24	4.15	7.55	5.03
	19	3.98	2.65	5.66	3.77	6.17	4.10	6.76	4.50	6.93	4.61	7.81	5.20
	20	4.26	2.83	5.80	3.86	6.84	4.55	6.97	4.64	7.68	5.11	8.08	5.38
	22	4.91	3.27	6.09	4.05	8.27	5.50	7.44	4.95	9.29	6.18	8.68	5.78
	24	5.76	3.83	6.41	4.26	9.84	6.55	7.99	5.32	11.1	7.36	9.65	6.42
	26	6.76	4.50	6.76	4.50	11.6	7.69	8.82	5.87	13.0	8.64	10.7	7.11
	28	7.84	5.22	7.16	4.76	13.4	8.91	9.65	6.42	15.1	10.0	11.7	7.80
	30	9.01	5.99	7.80	5.19	15.4	10.2	10.5	6.97	17.3	11.5	12.7	8.48
	32	10.2	6.82	8.47	5.64	17.5	11.6	11.3	7.52	19.7	13.1	13.8	9.16
	34	11.6	7.70	9.14	6.08								
	36	13.0	8.63	9.80	6.52								
	38	14.4	9.61	10.5	6.96								
	40	16.0	10.7	11.1	7.40								

Other Constants and Properties

	53		50		45	
$b_y \times 10^3$ (kip-ft)$^{-1}$	12.2	8.15	16.7	11.1	18.8	12.5
$t_y \times 10^3$ (kips)$^{-1}$	2.14	1.43	2.28	1.52	2.54	1.69
$t_r \times 10^3$ (kips)$^{-1}$	2.64	1.76	2.81	1.87	3.13	2.08
r_x/r_y	2.11		2.64		2.64	

Note: Heavy line indicates Kl/r equal to or greater than 200.

W12

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W12×											
		40				35^c				30^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	2.86	1.90	6.25	4.16	3.24	2.16	6.96	4.63	3.94	2.62	8.27	5.50
	6	3.16	2.10	6.25	4.16	3.79	2.52	7.09	4.72	4.54	3.02	8.46	5.63
	7	3.28	2.18	6.27	4.17	4.02	2.67	7.34	4.88	4.79	3.19	8.79	5.85
	8	3.42	2.28	6.44	4.29	4.29	2.86	7.61	5.06	5.10	3.39	9.14	6.08
	9	3.59	2.39	6.62	4.40	4.63	3.08	7.90	5.26	5.49	3.66	9.53	6.34
	10	3.78	2.52	6.80	4.53	5.04	3.35	8.21	5.46	5.99	3.99	9.94	6.62
	11	4.01	2.67	7.00	4.66	5.53	3.68	8.55	5.69	6.59	4.39	10.4	6.92
	12	4.28	2.85	7.21	4.79	6.12	4.07	8.92	5.93	7.32	4.87	10.9	7.25
	13	4.59	3.05	7.43	4.94	6.84	4.55	9.32	6.20	8.20	5.46	11.5	7.62
	14	4.95	3.29	7.66	5.10	7.71	5.13	9.75	6.49	9.28	6.17	12.1	8.02
	15	5.37	3.57	7.91	5.26	8.79	5.85	10.2	6.81	10.6	7.06	12.7	8.47
	16	5.85	3.89	8.18	5.44	10.0	6.66	10.8	7.16	12.1	8.03	13.7	9.12
	17	6.42	4.27	8.46	5.63	11.3	7.51	11.5	7.63	13.6	9.07	15.0	10.0
	18	7.08	4.71	8.77	5.83	12.7	8.42	12.4	8.27	15.3	10.2	16.3	10.9
	19	7.87	5.24	9.09	6.05	14.1	9.39	13.4	8.91	17.0	11.3	17.7	11.8
	20	8.72	5.80	9.45	6.29	15.6	10.4	14.4	9.56	18.9	12.5	19.0	12.7
	22	10.5	7.02	10.5	6.96	18.9	12.6	16.3	10.8	22.8	15.2	21.7	14.4
	24	12.6	8.35	11.8	7.82	22.5	15.0	18.2	12.1	27.2	18.1	24.4	16.2
	26	14.7	9.80	13.1	8.69								
	28	17.1	11.4	14.4	9.56								
	30	19.6	13.1	15.7	10.4								
	32	22.3	14.9	16.9	11.3								

Other Constants and Properties

	40		35^c		30^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	21.2	14.1	31.0	20.6	37.3	24.8
$t_y \times 10^3$ (kips)$^{-1}$	2.85	1.90	3.23	2.15	3.79	2.53
$t_r \times 10^3$ (kips)$^{-1}$	3.51	2.34	3.97	2.65	4.67	3.11
r_x/r_y	2.64		3.41		3.43	

[c] Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W12

Shape		W12×											
		26^c				22^c				19^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	4.66	3.10	9.58	6.37	5.41	3.60	12.2	8.09	6.52	4.34	14.4	9.60
	1	4.68	3.11	9.58	6.37	5.48	3.64	12.2	8.09	6.60	4.39	14.4	9.60
	2	4.73	3.15	9.58	6.37	5.67	3.77	12.2	8.09	6.84	4.55	14.4	9.60
	3	4.82	3.21	9.58	6.37	6.03	4.01	12.2	8.09	7.27	4.84	14.5	9.66
	4	4.95	3.30	9.58	6.37	6.58	4.38	13.0	8.64	7.95	5.29	15.6	10.4
	5	5.13	3.41	9.58	6.37	7.43	4.94	13.9	9.27	8.96	5.96	16.9	11.2
	6	5.36	3.57	9.83	6.54	8.73	5.81	15.0	10.0	10.5	6.99	18.4	12.2
	7	5.65	3.76	10.2	6.81	10.6	7.02	16.3	10.9	12.9	8.56	20.1	13.4
	8	6.00	3.99	10.7	7.11	13.2	8.75	17.8	11.9	16.3	10.8	22.3	14.8
	9	6.44	4.28	11.2	7.43	16.6	11.1	19.7	13.1	20.6	13.7	25.7	17.1
	10	6.98	4.64	11.7	7.78	20.6	13.7	22.9	15.2	25.4	16.9	30.4	20.2
	11	7.64	5.08	12.3	8.17	24.9	16.5	26.3	17.5	30.8	20.5	35.2	23.4
	12	8.49	5.65	12.9	8.60	29.6	19.7	29.8	19.8	36.6	24.4	40.0	26.6
	13	9.53	6.34	13.6	9.08	34.7	23.1	33.3	22.2	43.0	28.6	45.0	29.9
	14	10.8	7.19	14.4	9.61	40.3	26.8	36.9	24.5				
	15	12.4	8.23	15.4	10.3								
	16	14.1	9.36	17.1	11.4								
	17	15.9	10.6	18.8	12.5								
	18	17.8	11.8	20.6	13.7								
	19	19.8	13.2	22.3	14.8								
	20	22.0	14.6	24.1	16.0								
	21	24.2	16.1	25.9	17.2								
	22	26.6	17.7	27.6	18.4								
	23	29.1	19.3	29.4	19.6								
	24	31.7	21.1	31.2	20.8								
	25	34.3	22.9	33.0	22.0								

Other Constants and Properties

	26^c ASD	26^c LRFD	22^c ASD	22^c LRFD	19^c ASD	19^c LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	43.6	29.0	97.3	64.8	120	79.5
$t_y \times 10^3$ (kips)$^{-1}$	4.36	2.91	5.14	3.43	5.98	3.99
$t_r \times 10^3$ (kips)$^{-1}$	5.37	3.58	6.33	4.22	7.36	4.91
r_x/r_y	3.42		5.79		5.86	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W12

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W12×							
		16^c				14^c			
Design		$p \times 10^3$		$b_x \times 10^3$		$p \times 10^3$		$b_x \times 10^3$	
		(kips)$^{-1}$		(kip-ft)$^{-1}$		(kips)$^{-1}$		(kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	7.97	5.30	17.7	11.8	9.40	6.25	20.5	13.6
	1	8.07	5.37	17.7	11.8	9.52	6.33	20.5	13.6
	2	8.39	5.58	17.7	11.8	9.89	6.58	20.5	13.6
	3	8.96	5.96	18.1	12.0	10.6	7.03	21.0	14.0
	4	9.86	6.56	19.6	13.1	11.6	7.74	22.9	15.2
	5	11.3	7.49	21.4	14.3	13.3	8.84	25.1	16.7
	6	13.4	8.90	23.6	15.7	15.9	10.6	27.8	18.5
	7	16.8	11.2	26.3	17.5	20.0	13.3	31.1	20.7
	8	21.8	14.5	29.6	19.7	26.0	17.3	36.3	24.1
	9	27.6	18.3	36.1	24.0	32.9	21.9	44.5	29.6
	10	34.0	22.6	43.0	28.6	40.7	27.1	53.2	35.4
	11	41.2	27.4	50.1	33.3	49.2	32.7	62.3	41.4
	12	49.0	32.6	57.4	38.2	58.5	39.0	71.6	47.6

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	158	105	188	125
$t_y \times 10^3$ (kips)$^{-1}$	7.07	4.72	8.02	5.35
$t_r \times 10^3$ (kips)$^{-1}$	8.70	5.80	9.87	6.58
r_x/r_y	6.04		6.14	

[c] Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W10

Shape		W10×											
		112				100				88			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.01	0.675	2.42	1.61	1.14	0.756	2.74	1.82	1.29	0.858	3.15	2.10
	6	1.07	0.711	2.42	1.61	1.20	0.798	2.74	1.82	1.36	0.906	3.15	2.10
	7	1.09	0.725	2.42	1.61	1.22	0.813	2.74	1.82	1.39	0.924	3.15	2.10
	8	1.11	0.741	2.42	1.61	1.25	0.832	2.74	1.82	1.42	0.946	3.15	2.10
	9	1.14	0.760	2.42	1.61	1.28	0.853	2.74	1.82	1.46	0.971	3.15	2.10
	10	1.17	0.781	2.43	1.62	1.32	0.878	2.75	1.83	1.50	0.999	3.17	2.11
	11	1.21	0.806	2.45	1.63	1.36	0.906	2.78	1.85	1.55	1.03	3.20	2.13
	12	1.25	0.833	2.47	1.64	1.41	0.938	2.80	1.86	1.61	1.07	3.23	2.15
	13	1.30	0.864	2.49	1.66	1.46	0.974	2.82	1.88	1.67	1.11	3.27	2.17
	14	1.35	0.899	2.51	1.67	1.52	1.01	2.85	1.90	1.74	1.16	3.30	2.19
	15	1.41	0.938	2.53	1.68	1.59	1.06	2.87	1.91	1.82	1.21	3.33	2.22
	16	1.48	0.982	2.54	1.69	1.67	1.11	2.90	1.93	1.90	1.27	3.36	2.24
	17	1.55	1.03	2.56	1.71	1.75	1.17	2.92	1.95	2.00	1.33	3.40	2.26
	18	1.63	1.08	2.58	1.72	1.85	1.23	2.95	1.96	2.11	1.41	3.43	2.28
	19	1.72	1.15	2.60	1.73	1.95	1.30	2.98	1.98	2.23	1.49	3.47	2.31
	20	1.82	1.21	2.63	1.75	2.07	1.38	3.00	2.00	2.37	1.58	3.50	2.33
	22	2.06	1.37	2.67	1.77	2.35	1.56	3.06	2.03	2.69	1.79	3.58	2.38
	24	2.36	1.57	2.71	1.80	2.69	1.79	3.12	2.07	3.10	2.06	3.65	2.43
	26	2.74	1.82	2.76	1.83	3.14	2.09	3.17	2.11	3.62	2.41	3.73	2.48
	28	3.17	2.11	2.80	1.86	3.64	2.42	3.24	2.15	4.19	2.79	3.82	2.54
	30	3.64	2.42	2.85	1.90	4.18	2.78	3.30	2.20	4.81	3.20	3.91	2.60
	32	4.15	2.76	2.90	1.93	4.75	3.16	3.37	2.24	5.48	3.64	4.00	2.66
	34	4.68	3.11	2.95	1.96	5.36	3.57	3.44	2.29	6.18	4.11	4.10	2.73
	36	5.25	3.49	3.01	2.00	6.01	4.00	3.51	2.33	6.93	4.61	4.20	2.79
	38	5.85	3.89	3.06	2.04	6.70	4.46	3.58	2.38	7.72	5.14	4.30	2.86
	40	6.48	4.31	3.12	2.07	7.42	4.94	3.66	2.44	8.56	5.69	4.42	2.94

Other Constants and Properties

	112		100		88	
$b_y \times 10^3$ (kip-ft)$^{-1}$	5.15	3.43	5.84	3.89	6.71	4.46
$t_y \times 10^3$ (kips)$^{-1}$	1.01	0.675	1.13	0.756	1.29	0.858
$t_r \times 10^3$ (kips)$^{-1}$	1.25	0.830	1.40	0.930	1.58	1.06
r_x/r_y	1.74		1.74		1.73	

W10

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W10×											
		77				68				60			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.48	0.982	3.65	2.43	1.67	1.11	4.18	2.78	1.89	1.26	4.78	3.18
	6	1.56	1.04	3.65	2.43	1.77	1.18	4.18	2.78	2.00	1.33	4.78	3.18
	7	1.59	1.06	3.65	2.43	1.81	1.20	4.18	2.78	2.05	1.36	4.78	3.18
	8	1.63	1.08	3.65	2.43	1.85	1.23	4.18	2.78	2.10	1.39	4.78	3.18
	9	1.67	1.11	3.65	2.43	1.90	1.26	4.18	2.78	2.15	1.43	4.78	3.18
	10	1.72	1.15	3.68	2.45	1.96	1.30	4.22	2.81	2.22	1.48	4.84	3.22
	11	1.78	1.19	3.72	2.48	2.02	1.34	4.27	2.84	2.30	1.53	4.90	3.26
	12	1.85	1.23	3.76	2.50	2.10	1.39	4.33	2.88	2.38	1.58	4.97	3.31
	13	1.92	1.28	3.80	2.53	2.18	1.45	4.38	2.91	2.48	1.65	5.04	3.36
	14	2.00	1.33	3.85	2.56	2.27	1.51	4.44	2.95	2.59	1.72	5.12	3.41
	15	2.09	1.39	3.89	2.59	2.38	1.58	4.49	2.99	2.71	1.80	5.19	3.46
	16	2.20	1.46	3.94	2.62	2.50	1.66	4.55	3.03	2.85	1.89	5.27	3.51
	17	2.31	1.54	3.98	2.65	2.63	1.75	4.61	3.07	3.00	2.00	5.35	3.56
	18	2.44	1.63	4.03	2.68	2.78	1.85	4.68	3.11	3.17	2.11	5.43	3.62
	19	2.59	1.72	4.08	2.71	2.95	1.96	4.74	3.15	3.37	2.24	5.52	3.67
	20	2.75	1.83	4.13	2.74	3.13	2.08	4.81	3.20	3.58	2.38	5.61	3.73
	22	3.14	2.09	4.23	2.81	3.57	2.38	4.94	3.29	4.09	2.72	5.79	3.85
	24	3.62	2.41	4.33	2.88	4.13	2.75	5.09	3.39	4.74	3.15	5.99	3.99
	26	4.23	2.82	4.45	2.96	4.83	3.22	5.24	3.49	5.56	3.70	6.20	4.13
	28	4.91	3.27	4.56	3.04	5.60	3.73	5.41	3.60	6.44	4.29	6.43	4.28
	30	5.63	3.75	4.69	3.12	6.43	4.28	5.58	3.71	7.40	4.92	6.67	4.44
	32	6.41	4.26	4.82	3.21	7.32	4.87	5.77	3.84	8.42	5.60	6.94	4.61
	34	7.24	4.81	4.96	3.30	8.26	5.50	5.96	3.97	9.50	6.32	7.22	4.80
	36	8.11	5.40	5.11	3.40	9.26	6.16	6.18	4.11	10.7	7.09	7.53	5.01
	38	9.04	6.01	5.27	3.50	10.3	6.87	6.40	4.26	11.9	7.90	7.96	5.30
	40	10.0	6.66	5.43	3.61	11.4	7.61	6.65	4.43	13.2	8.75	8.43	5.61

Other Constants and Properties

$b_y \times 10^3$ (kip-ft)$^{-1}$	7.76	5.16	8.88	5.91	10.2	6.77
$t_y \times 10^3$ (kips)$^{-1}$	1.47	0.982	1.67	1.11	1.89	1.26
$t_r \times 10^3$ (kips)$^{-1}$	1.81	1.21	2.05	1.37	2.33	1.55
r_x/r_y	1.73		1.71		1.71	

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W10

Shape		W10×											
		54				49				45			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	**2.11**	1.40	**5.35**	3.56	**2.32**	1.54	**5.90**	3.92	**2.52**	1.68	**6.49**	4.32
	6	**2.24**	1.49	**5.35**	3.56	**2.46**	1.63	**5.90**	3.92	**2.77**	1.84	**6.49**	4.32
	7	**2.28**	1.52	**5.35**	3.56	**2.51**	1.67	**5.90**	3.92	**2.86**	1.90	**6.49**	4.32
	8	**2.34**	1.56	**5.35**	3.56	**2.57**	1.71	**5.90**	3.92	**2.98**	1.98	**6.60**	4.39
	9	**2.40**	1.60	**5.35**	3.56	**2.64**	1.76	**5.90**	3.93	**3.11**	2.07	**6.73**	4.48
	10	**2.48**	1.65	**5.43**	3.61	**2.73**	1.81	**6.00**	3.99	**3.27**	2.17	**6.87**	4.57
	11	**2.56**	1.71	**5.51**	3.67	**2.82**	1.88	**6.10**	4.06	**3.45**	2.30	**7.01**	4.66
	12	**2.66**	1.77	**5.60**	3.72	**2.93**	1.95	**6.20**	4.13	**3.67**	2.44	**7.15**	4.76
	13	**2.77**	1.84	**5.69**	3.78	**3.05**	2.03	**6.31**	4.20	**3.91**	2.60	**7.30**	4.86
	14	**2.89**	1.92	**5.78**	3.84	**3.19**	2.12	**6.42**	4.27	**4.20**	2.79	**7.46**	4.96
	15	**3.03**	2.02	**5.87**	3.91	**3.34**	2.22	**6.54**	4.35	**4.53**	3.01	**7.63**	5.07
	16	**3.18**	2.12	**5.97**	3.97	**3.52**	2.34	**6.66**	4.43	**4.91**	3.27	**7.80**	5.19
	17	**3.36**	2.23	**6.07**	4.04	**3.71**	2.47	**6.78**	4.51	**5.35**	3.56	**7.98**	5.31
	18	**3.55**	2.36	**6.18**	4.11	**3.93**	2.61	**6.91**	4.60	**5.86**	3.90	**8.17**	5.44
	19	**3.77**	2.51	**6.29**	4.18	**4.17**	2.78	**7.04**	4.69	**6.46**	4.30	**8.37**	5.57
	20	**4.01**	2.67	**6.40**	4.26	**4.45**	2.96	**7.18**	4.78	**7.15**	4.76	**8.58**	5.71
	22	**4.59**	3.06	**6.64**	4.41	**5.10**	3.39	**7.48**	4.98	**8.66**	5.76	**9.03**	6.01
	24	**5.32**	3.54	**6.89**	4.58	**5.93**	3.94	**7.80**	5.19	**10.3**	6.85	**9.53**	6.34
	26	**6.24**	4.15	**7.17**	4.77	**6.96**	4.63	**8.15**	5.42	**12.1**	8.04	**10.1**	6.71
	28	**7.24**	4.82	**7.47**	4.97	**8.07**	5.37	**8.53**	5.68	**14.0**	9.33	**10.9**	7.23
	30	**8.31**	5.53	**7.79**	5.18	**9.26**	6.16	**8.95**	5.96	**16.1**	10.7	**11.8**	7.82
	32	**9.46**	6.29	**8.15**	5.42	**10.5**	7.01	**9.47**	6.30	**18.3**	12.2	**12.6**	8.41
	34	**10.7**	7.10	**8.57**	5.70	**11.9**	7.92	**10.2**	6.77				
	36	**12.0**	7.96	**9.15**	6.09	**13.3**	8.88	**10.9**	7.24				
	38	**13.3**	8.87	**9.73**	6.48	**14.9**	9.89	**11.6**	7.71				
	40	**14.8**	9.83	**10.3**	6.86	**16.5**	11.0	**12.3**	8.18				

Other Constants and Properties

	54		49		45	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**11.4**	7.57	**12.6**	8.38	**17.6**	11.7
$t_y \times 10^3$ (kips)$^{-1}$	**2.11**	1.40	**2.31**	1.54	**2.51**	1.68
$t_r \times 10^3$ (kips)$^{-1}$	**2.59**	1.73	**2.84**	1.90	**3.09**	2.06
r_x/r_y	1.71		1.71		2.15	

Note: Heavy line indicates *Kl*/*r* equal to or greater than 200.

W10

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W10×											
		39				33				30			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	2.91	1.94	7.61	5.06	3.44	2.29	9.18	6.11	3.78	2.51	9.73	6.48
	6	3.21	2.13	7.61	5.06	3.80	2.53	9.18	6.11	4.62	3.08	10.1	6.74
	7	3.32	2.21	7.61	5.07	3.95	2.63	9.22	6.13	4.97	3.31	10.5	6.99
	8	3.46	2.30	7.78	5.18	4.11	2.74	9.45	6.29	5.41	3.60	10.9	7.25
	9	3.62	2.41	7.96	5.29	4.31	2.87	9.70	6.45	5.95	3.96	11.3	7.53
	10	3.81	2.53	8.14	5.41	4.55	3.03	10.0	6.62	6.62	4.41	11.8	7.84
	11	4.03	2.68	8.33	5.54	4.83	3.21	10.2	6.80	7.45	4.96	12.3	8.17
	12	4.29	2.85	8.53	5.68	5.15	3.42	10.5	7.00	8.48	5.64	12.8	8.54
	13	4.58	3.05	8.74	5.81	5.52	3.67	10.8	7.20	9.76	6.49	13.4	8.93
	14	4.93	3.28	8.96	5.96	5.95	3.96	11.1	7.42	11.3	7.53	14.1	9.37
	15	5.33	3.55	9.19	6.12	6.46	4.30	11.5	7.64	13.0	8.65	14.8	9.85
	16	5.79	3.85	9.44	6.28	7.04	4.68	11.9	7.89	14.8	9.84	15.6	10.4
	17	6.33	4.21	9.70	6.45	7.72	5.14	12.2	8.15	16.7	11.1	16.8	11.2
	18	6.95	4.63	10.0	6.63	8.52	5.67	12.7	8.42	18.7	12.4	18.1	12.1
	19	7.69	5.12	10.3	6.82	9.46	6.30	13.1	8.72	20.8	13.9	19.4	12.9
	20	8.52	5.67	10.6	7.03	10.5	6.98	13.6	9.04	23.1	15.4	20.7	13.7
	22	10.3	6.86	11.2	7.47	12.7	8.44	14.7	9.80	28.0	18.6	23.2	15.4
	24	12.3	8.17	12.0	7.98	15.1	10.0	16.5	11.0				
	26	14.4	9.58	13.2	8.77	17.7	11.8	18.3	12.2				
	28	16.7	11.1	14.4	9.59	20.6	13.7	20.1	13.3				
	30	19.2	12.8	15.6	10.4	23.6	15.7	21.8	14.5				
	32	21.8	14.5	16.8	11.2	26.8	17.9	23.6	15.7				

Other Constants and Properties

	39		33		30	
$b_y \times 10^3$ (kip-ft)$^{-1}$	20.7	13.8	25.4	16.9	40.3	26.8
$t_y \times 10^3$ (kips)$^{-1}$	2.91	1.94	3.43	2.29	3.77	2.51
$t_r \times 10^3$ (kips)$^{-1}$	3.58	2.39	4.23	2.82	4.64	3.09
r_x/r_y	2.16		2.16		3.20	

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W10

Shape		W10×											
		26				22^c				19			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	4.39	2.92	11.4	7.57	5.19	3.45	13.7	9.12	5.95	3.96	16.5	11.0
	1	4.41	2.94	11.4	7.57	5.22	3.47	13.7	9.12	6.03	4.01	16.5	11.0
	2	4.49	2.99	11.4	7.57	5.30	3.53	13.7	9.12	6.28	4.18	16.5	11.0
	3	4.62	3.07	11.4	7.57	5.44	3.62	13.7	9.12	6.73	4.48	16.5	11.0
	4	4.81	3.20	11.4	7.57	5.66	3.77	13.7	9.12	7.42	4.93	17.4	11.6
	5	5.06	3.37	11.5	7.63	5.98	3.98	13.9	9.23	8.39	5.59	18.6	12.4
	6	5.39	3.58	11.9	7.93	6.38	4.25	14.5	9.63	9.77	6.50	19.9	13.2
	7	5.80	3.86	12.4	8.25	6.89	4.59	15.1	10.1	11.7	7.78	21.4	14.3
	8	6.32	4.20	12.9	8.59	7.54	5.02	15.9	10.6	14.4	9.56	23.2	15.4
	9	6.96	4.63	13.5	8.97	8.34	5.55	16.7	11.1	18.1	12.0	25.3	16.8
	10	7.75	5.16	14.1	9.39	9.34	6.21	17.5	11.7	22.3	14.9	28.2	18.8
	11	8.74	5.81	14.8	9.84	10.6	7.04	18.5	12.3	27.0	18.0	32.3	21.5
	12	10.0	6.63	15.5	10.3	12.1	8.07	19.6	13.0	32.2	21.4	36.4	24.2
	13	11.5	7.65	16.4	10.9	14.1	9.39	20.8	13.9	37.7	25.1	40.5	27.0
	14	13.3	8.87	17.3	11.5	16.4	10.9	22.4	14.9	43.8	29.1	44.7	29.7
	15	15.3	10.2	18.4	12.2	18.8	12.5	24.9	16.6				
	16	17.4	11.6	20.1	13.4	21.4	14.2	27.3	18.2				
	17	19.7	13.1	21.8	14.5	24.1	16.1	29.8	19.8				
	18	22.0	14.7	23.6	15.7	27.1	18.0	32.3	21.5				
	19	24.6	16.3	25.3	16.8	30.1	20.1	34.8	23.1				
	20	27.2	18.1	27.0	18.0	33.4	22.2	37.3	24.8				
	21	30.0	20.0	28.8	19.1	36.8	24.5	39.8	26.5				
	22	32.9	21.9	30.5	20.3	40.4	26.9	42.3	28.1				

Other Constants and Properties

	W10×26 ASD	W10×26 LRFD	W10×22 ASD	W10×22 LRFD	W10×19 ASD	W10×19 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	47.5	31.6	58.4	38.9	106	70.8
$t_y \times 10^3$ (kips)$^{-1}$	4.38	2.92	5.14	3.43	5.94	3.96
$t_r \times 10^3$ (kips)$^{-1}$	5.39	3.59	6.33	4.22	7.31	4.87
r_x/r_y	3.20		3.21		4.74	

^c Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

W10

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape		W10×											
		17^c				15^c				12^c			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	6.76	4.50	19.1	12.7	7.77	5.17	22.3	14.8	10.3	6.88	28.5	19.0
	1	6.85	4.55	19.1	12.7	7.87	5.23	22.3	14.8	10.5	6.97	28.5	19.0
	2	7.11	4.73	19.1	12.7	8.19	5.45	22.3	14.8	10.9	7.24	28.5	19.0
	3	7.64	5.08	19.1	12.7	8.76	5.83	22.5	15.0	11.6	7.74	28.8	19.1
	4	8.47	5.64	20.4	13.6	9.79	6.51	24.2	16.1	12.8	8.53	31.1	20.7
	5	9.67	6.44	21.9	14.6	11.3	7.53	26.1	17.4	14.6	9.74	33.9	22.6
	6	11.4	7.57	23.6	15.7	13.5	8.98	28.4	18.9	17.5	11.6	37.3	24.8
	7	13.8	9.17	25.7	17.1	16.6	11.1	31.2	20.7	21.8	14.5	41.3	27.5
	8	17.2	11.4	28.1	18.7	21.2	14.1	34.5	22.9	28.1	18.7	46.4	30.9
	9	21.8	14.5	31.0	20.6	26.8	17.8	39.7	26.4	35.6	23.7	56.5	37.6
	10	26.9	17.9	36.1	24.0	33.1	22.0	46.8	31.1	44.0	29.3	67.2	44.7
	11	32.5	21.6	41.6	27.7	40.1	26.7	54.1	36.0	53.2	35.4	78.3	52.1
	12	38.7	25.8	47.0	31.3	47.7	31.7	61.4	40.9	63.3	42.1	89.7	59.7
	13	45.4	30.2	52.6	35.0	55.9	37.2	68.9	45.8	74.3	49.4	101	67.3
	14	52.7	35.1	58.1	38.6								

Other Constants and Properties

	17^c		15^c		12^c	
$b_y \times 10^3$ (kip-ft)$^{-1}$	127	84.7	155	103	207	138
$t_y \times 10^3$ (kips)$^{-1}$	6.68	4.45	7.56	5.04	9.43	6.29
$t_r \times 10^3$ (kips)$^{-1}$	8.22	5.48	9.30	6.20	11.6	7.74
r_x/r_y	4.79		4.88		4.97	

^{c} Shape is slender for compression with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W8

Shape		W8×											
		67				58				48			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	1.70	1.13	5.08	3.38	1.95	1.30	5.96	3.96	2.37	1.58	7.27	4.84
	6	1.85	1.23	5.08	3.38	2.13	1.42	5.96	3.96	2.59	1.72	7.27	4.84
	7	1.90	1.27	5.08	3.38	2.20	1.46	5.96	3.96	2.67	1.78	7.27	4.84
	8	1.97	1.31	5.11	3.40	2.28	1.52	6.00	3.99	2.77	1.84	7.34	4.88
	9	2.05	1.37	5.16	3.43	2.37	1.58	6.07	4.04	2.89	1.92	7.44	4.95
	10	2.15	1.43	5.21	3.47	2.48	1.65	6.14	4.08	3.02	2.01	7.55	5.02
	11	2.25	1.50	5.26	3.50	2.61	1.74	6.21	4.13	3.18	2.12	7.66	5.09
	12	2.38	1.58	5.32	3.54	2.76	1.83	6.29	4.18	3.36	2.24	7.77	5.17
	13	2.52	1.68	5.37	3.58	2.93	1.95	6.36	4.23	3.57	2.38	7.88	5.25
	14	2.69	1.79	5.43	3.61	3.12	2.08	6.44	4.29	3.82	2.54	8.00	5.32
	15	2.87	1.91	5.49	3.65	3.35	2.23	6.52	4.34	4.10	2.73	8.13	5.41
	16	3.09	2.06	5.55	3.69	3.60	2.40	6.61	4.39	4.42	2.94	8.25	5.49
	17	3.34	2.22	5.61	3.73	3.90	2.59	6.69	4.45	4.79	3.19	8.38	5.58
	18	3.63	2.41	5.67	3.77	4.24	2.82	6.78	4.51	5.21	3.47	8.52	5.67
	19	3.95	2.63	5.73	3.82	4.63	3.08	6.87	4.57	5.70	3.80	8.66	5.76
	20	4.33	2.88	5.80	3.86	5.09	3.38	6.96	4.63	6.28	4.18	8.80	5.86
	22	5.24	3.49	5.93	3.95	6.15	4.09	7.15	4.76	7.60	5.06	9.11	6.06
	24	6.24	4.15	6.07	4.04	7.32	4.87	7.35	4.89	9.05	6.02	9.43	6.27
	26	7.32	4.87	6.22	4.14	8.60	5.72	7.56	5.03	10.6	7.07	9.78	6.51
	28	8.49	5.65	6.37	4.24	9.97	6.63	7.79	5.18	12.3	8.19	10.2	6.76
	30	9.75	6.49	6.53	4.35	11.4	7.61	8.03	5.34	14.1	9.41	10.6	7.03
	32	11.1	7.38	6.71	4.46	13.0	8.66	8.28	5.51	16.1	10.7	11.0	7.32
	34	12.5	8.33	6.88	4.58	14.7	9.78	8.56	5.69	18.2	12.1	11.5	7.64

Other Constants and Properties

	67		58		48	
$b_y \times 10^3$ (kip-ft)$^{-1}$	10.9	7.25	12.8	8.50	15.6	10.4
$t_y \times 10^3$ (kips)$^{-1}$	1.69	1.13	1.95	1.30	2.36	1.58
$t_r \times 10^3$ (kips)$^{-1}$	2.09	1.39	2.40	1.60	2.91	1.94
r_x/r_y	1.75		1.74		1.74	

Note: Heavy line indicates Kl/r equal to or greater than 200.

W8

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape	W8×							
	40				35			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length *KL* (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending								
0	2.85	1.89	8.95	5.96	3.25	2.16	10.3	6.83
6	3.12	2.07	8.95	5.96	3.56	2.37	10.3	6.83
7	3.22	2.14	8.95	5.96	3.68	2.45	10.3	6.83
8	3.35	2.23	9.07	6.04	3.82	2.54	10.4	6.94
9	3.49	2.32	9.23	6.14	3.99	2.66	10.6	7.07
10	3.66	2.44	9.39	6.24	4.19	2.79	10.8	7.21
11	3.86	2.57	9.55	6.35	4.42	2.94	11.1	7.36
12	4.10	2.73	9.72	6.47	4.69	3.12	11.3	7.51
13	4.36	2.90	9.90	6.59	5.00	3.33	11.5	7.66
14	4.67	3.11	10.1	6.71	5.36	3.57	11.8	7.83
15	5.03	3.35	10.3	6.84	5.77	3.84	12.0	8.00
16	5.44	3.62	10.5	6.97	6.25	4.16	12.3	8.18
17	5.91	3.93	10.7	7.11	6.80	4.52	12.6	8.37
18	6.46	4.30	10.9	7.25	7.43	4.95	12.9	8.56
19	7.09	4.72	11.1	7.40	8.17	5.44	13.2	8.77
20	7.84	5.22	11.4	7.56	9.04	6.02	13.5	8.98
22	9.49	6.32	11.9	7.89	10.9	7.28	14.2	9.44
24	11.3	7.52	12.4	8.25	13.0	8.66	15.0	9.96
26	13.3	8.82	13.0	8.65	15.3	10.2	15.8	10.5
28	15.4	10.2	13.6	9.08	17.7	11.8	17.0	11.3
30	17.7	11.7	14.4	9.58	20.3	13.5	18.4	12.2
32	20.1	13.4	15.5	10.3	23.2	15.4	19.8	13.2
34	22.7	15.1	16.5	11.0				

Other Constants and Properties

	40		35	
$b_y \times 10^3$ (kip-ft)$^{-1}$	19.3	12.8	22.1	14.7
$t_y \times 10^3$ (kips)$^{-1}$	2.84	1.89	3.24	2.16
$t_r \times 10^3$ (kips)$^{-1}$	3.50	2.33	3.99	2.66
r_x/r_y	1.73		1.73	

Note: Heavy line indicates *Kl/r* equal to or greater than 200.

F_y = 50 ksi

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

W8

Shape		W8×							
		31				28			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	0	3.66	2.44	11.7	7.80	4.05	2.70	13.1	8.71
	6	4.02	2.67	11.7	7.80	4.68	3.11	13.2	8.77
	7	4.16	2.76	11.7	7.80	4.93	3.28	13.5	9.00
	8	4.32	2.87	11.9	7.94	5.24	3.49	13.9	9.23
	9	4.51	3.00	12.2	8.11	5.61	3.73	14.2	9.48
	10	4.74	3.15	12.5	8.29	6.05	4.03	14.6	9.74
	11	5.00	3.33	12.7	8.48	6.58	4.38	15.0	10.00
	12	5.31	3.53	13.0	8.67	7.22	4.80	15.5	10.3
	13	5.66	3.77	13.3	8.88	7.98	5.31	15.9	10.6
	14	6.07	4.04	13.7	9.09	8.90	5.92	16.4	10.9
	15	6.54	4.35	14.0	9.32	9.99	6.65	17.0	11.3
	16	7.09	4.72	14.4	9.56	11.3	7.54	17.5	11.7
	17	7.72	5.14	14.7	9.81	12.8	8.52	18.1	12.0
	18	8.45	5.62	15.1	10.1	14.3	9.55	18.7	12.5
	19	9.30	6.18	15.6	10.3	16.0	10.6	19.4	12.9
	20	10.3	6.85	16.0	10.6	17.7	11.8	20.2	13.4
	22	12.5	8.29	17.0	11.3	21.4	14.3	22.2	14.7
	24	14.8	9.87	18.0	12.0	25.5	17.0	24.5	16.3
	26	17.4	11.6	19.6	13.1	29.9	19.9	26.9	17.9
	28	20.2	13.4	21.4	14.3				
	30	23.2	15.4	23.3	15.5				
	32	26.4	17.5	25.1	16.7				

Other Constants and Properties

	W8×31 ASD	W8×31 LRFD	W8×28 ASD	W8×28 LRFD
$b_y \times 10^3$ (kip-ft)$^{-1}$	25.3	16.8	35.3	23.5
$t_y \times 10^3$ (kips)$^{-1}$	3.65	2.44	4.04	2.70
$t_r \times 10^3$ (kips)$^{-1}$	4.50	3.00	4.98	3.32
r_x/r_y	1.72		2.13	

Note: Heavy line indicates Kl/r equal to or greater than 200.

W8

Table 6–1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

Shape	W8×											
	24				21				18			
Design	$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
0	4.72	3.14	15.4	10.3	5.42	3.61	17.5	11.6	6.35	4.22	21.0	13.9
1	4.74	3.15	15.4	10.3	5.46	3.63	17.5	11.6	6.39	4.25	21.0	13.9
2	4.80	3.19	15.4	10.3	5.56	3.70	17.5	11.6	6.53	4.34	21.0	13.9
3	4.90	3.26	15.4	10.3	5.75	3.83	17.5	11.6	6.76	4.50	21.0	13.9
4	5.04	3.35	15.4	10.3	6.03	4.01	17.5	11.6	7.09	4.72	21.0	13.9
5	5.22	3.48	15.4	10.3	6.40	4.26	17.8	11.9	7.55	5.03	21.5	14.3
6	5.46	3.63	15.6	10.4	6.88	4.58	18.5	12.3	8.15	5.42	22.5	15.0
7	5.76	3.83	16.0	10.6	7.50	4.99	19.3	12.8	8.93	5.94	23.5	15.6
8	6.12	4.07	16.5	11.0	8.28	5.51	20.1	13.3	9.91	6.59	24.6	16.4
9	6.56	4.36	17.0	11.3	9.27	6.17	20.9	13.9	11.2	7.42	25.9	17.2
10	7.09	4.71	17.5	11.6	10.5	7.00	21.9	14.6	12.7	8.47	27.3	18.1
11	7.72	5.13	18.1	12.0	12.1	8.04	22.9	15.3	14.7	9.80	28.8	19.2
12	8.47	5.64	18.7	12.4	14.1	9.38	24.1	16.0	17.3	11.5	30.5	20.3
13	9.38	6.24	19.3	12.9	16.5	11.0	25.4	16.9	20.3	13.5	32.5	21.6
14	10.5	6.96	20.0	13.3	19.2	12.8	26.8	17.8	23.6	15.7	35.3	23.5
15	11.8	7.83	20.7	13.8	22.0	14.7	28.6	19.0	27.1	18.0	38.8	25.8
16	13.4	8.90	21.5	14.3	25.1	16.7	31.1	20.7	30.8	20.5	42.4	28.2
17	15.1	10.0	22.4	14.9	28.3	18.8	33.5	22.3	34.8	23.1	45.9	30.5
18	16.9	11.3	23.3	15.5	31.7	21.1	36.0	24.0	39.0	25.9	49.4	32.9
19	18.9	12.5	24.4	16.2	35.3	23.5	38.5	25.6	43.4	28.9	52.9	35.2
20	20.9	13.9	26.0	17.3	39.2	26.1	40.9	27.2	48.1	32.0	56.4	37.5
21	23.0	15.3	27.6	18.4	43.2	28.7	43.3	28.8				
22	25.3	16.8	29.3	19.5								
23	27.6	18.4	30.9	20.5								
24	30.1	20.0	32.5	21.6								
25	32.6	21.7	34.1	22.7								

Other Constants and Properties

	W8×24		W8×21		W8×18	
$b_y \times 10^3$ (kip-ft)$^{-1}$	41.6	27.7	62.6	41.7	76.5	50.9
$t_y \times 10^3$ (kips)$^{-1}$	4.71	3.14	5.41	3.61	6.33	4.22
$t_r \times 10^3$ (kips)$^{-1}$	5.80	3.87	6.66	4.44	7.80	5.20
r_x/r_y	2.12		2.77		2.79	

Note: Heavy line indicates Kl/r equal to or greater than 200.

Table 6-1 (continued)
Combined Axial and Bending
W Shapes

F_y = 50 ksi

W8

Shape		W8×											
		15				13				10^{c,f}			
Design		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$		$p \times 10^3$ (kips)$^{-1}$		$b_x \times 10^3$ (kip-ft)$^{-1}$	
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
Effective length KL (ft) with respect to least radius of gyration r_y or Unbraced Length L_b (ft) for X-X axis bending	**0**	7.52	5.01	**26.2**	17.4	8.70	5.79	**31.3**	20.8	11.7	7.77	**40.6**	27.0
	1	7.63	5.07	**26.2**	17.4	8.83	5.88	**31.3**	20.8	11.8	7.86	**40.6**	27.0
	2	7.95	5.29	**26.2**	17.4	9.23	6.14	**31.3**	20.8	12.3	8.16	**40.6**	27.0
	3	8.51	5.66	**26.2**	17.4	9.94	6.62	**31.3**	20.8	13.1	8.70	**40.6**	27.0
	4	9.37	6.23	**27.6**	18.4	11.0	7.34	**33.4**	22.2	14.3	9.53	**43.2**	28.8
	5	10.6	7.05	**29.4**	19.5	12.6	8.39	**35.7**	23.8	16.3	10.9	**46.7**	31.0
	6	12.3	8.20	**31.3**	20.9	14.8	9.87	**38.5**	25.6	19.3	12.8	**50.7**	33.7
	7	14.7	9.81	**33.6**	22.4	18.0	12.0	**41.6**	27.7	23.4	15.5	**55.5**	36.9
	8	18.1	12.0	**36.2**	24.1	22.5	15.0	**45.4**	30.2	29.2	19.5	**61.3**	40.8
	9	22.8	15.2	**39.3**	26.1	28.5	18.9	**49.9**	33.2	37.0	24.6	**70.7**	47.1
	10	28.1	18.7	**42.9**	28.6	35.1	23.4	**57.2**	38.1	45.7	30.4	**83.6**	55.7
	11	34.0	22.6	**48.9**	32.5	42.5	28.3	**65.6**	43.7	55.3	36.8	**96.9**	64.4
	12	40.5	26.9	**54.9**	36.6	50.6	33.7	**74.0**	49.3	65.8	43.8	**110**	73.4
	13	47.5	31.6	**61.0**	40.6	59.4	39.5	**82.5**	54.9	77.2	51.4	**124**	82.5
	14	55.1	36.7	**67.0**	44.6	68.9	45.8	**90.9**	60.5	89.6	59.6	**138**	91.6

Other Constants and Properties

	15		13		10^{c,f}	
$b_y \times 10^3$ (kip-ft)$^{-1}$	**133**	88.8	**166**	110	**218**	145
$t_y \times 10^3$ (kips)$^{-1}$	**7.51**	5.01	**8.69**	5.79	**11.2**	7.50
$t_r \times 10^3$ (kips)$^{-1}$	**9.24**	6.16	**10.7**	7.13	**13.8**	9.23
r_x/r_y	3.76		3.81		3.83	

[c] Shape is slender for compression with F_y = 50 ksi.

[f] Shape does not meet compact limit for flexure with F_y = 50 ksi.

Note: Heavy line indicates Kl/r equal to or greater than 200.

PART 7

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of bolts in steel-to-steel structural connections. Additional guidance on bolt design is available in AISC Design Guide 17, *High Strength Bolts – A Primer for Structural Engineers*, (Kulak, 2002). For the design of steel-to-concrete anchorage, see Part 14. For the design of connection elements, see Part 9. For the design of simple shear, moment, bracing, and other connections, see Parts 10 through 15. For bolted joints that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

GENERAL REQUIREMENTS FOR BOLTED JOINTS

Fastener Components

The applicable material specifications for fastener components are as given in Part 2. Material and storage requirements fastener components are as given in AISC Specification Section A3.3 and RCSC Specification Section 2. The compatibility of ASTM A563 nuts and F436 washers with ASTM A325, F1852, and A490 bolts is as given in RCSC Specification Table 2.1. These products are given identifying marks, as illustrated in RCSC Specification Figure C-2.1. Alternative-design fasteners, including twist-off-type tension-control bolt assemblies with a strength level matching that of ASTM A490 bolts, and alternative washer-type indicating devices are permitted, subject to the requirements in RCSC Specification Sections 2.8 and 2.6.2, respectively.

Mixing grades of fasteners raises inventory and quality control issues associated with the use of multiple fastener grades. When both ASTM A325 and A490 bolts are used on a project, different diameters can be specified for each to help ensure that the ASTM A490 bolts are installed in the proper location.

Regardless of the bolt type selected, the typical sizes of $^3/_4$-in., $^7/_8$-in., 1-in. and 1$^1/_8$-in. diameter are usually preferred. Diameters above 1 in. require special consideration for availability as well as installation, when pretensioned installation is required. Special equipment may be required to pretension large-diameter ASTM A490 bolts.

Proper Selection of Bolt Length

Per RCSC Specification Section 2.3.2, adequate thread engagement is developed when the end of the bolt is at least flush with or projects beyond the face of the nut. To provide for this, the ordered length of ASTM A325, F1852, and A490 bolts should be calculated as the grip (see Figure 7-1) plus the nominal thickness of washers and/or direct-tension indicators, if used, plus the allowance from Table 7-15, with the total rounded to the next higher increment of $^1/_4$ in. up to a 5 in. length and the next higher $^1/_2$ in. over a 5 in. length. Note that bolts longer than five inches are generally available only in $^1/_2$-in. increments, except by special arrangement with the manufacturer or vendor. While longer lengths may be ordered, an 8-in. length is generally the maximum stock length available. Requirements for a minimum stick-through greater than zero are discouraged because of the risk of jamming the nut on the thread runout, particularly in the bolt length range available only in $^1/_2$-in. increments. See Carter (1996) for further information.

Washer Requirements

Requirements for the use of ASTM F436 washers and/or plate washers are given in RCSC Specification Section 6.

Nut Requirements

The compatibility of ASTM A563 nuts with ASTM A325, F1852, and A490 bolts is as given in RCSC Specification Table 2.1.

Bolted Parts

The requirements for connected plies, faying surfaces, bolt holes, and burrs are given in AISC Specification Sections J3.2 and M2.5, and RCSC Specification Section 3. Spacing and edge distance requirements are given in AISC Specification Sections J3.3, J3.4, and J3.5.

PROPER SPECIFICATION OF JOINT TYPE

When ASTM A325, F1852, or A490 high-strength bolts are to be used, the joint type must be specified as snug-tightened, pretensioned, or slip-critical, per RCSC Specification Section 4.

Snug-Tightened Joints

Snug-tightened joints simplify design, installation, and inspection and should be specified whenever pretensioned joints and slip-critical joints are not required. The applicability is summarized and design requirements, installation requirements, and inspection requirements are stipulated for snug-tightened joints per RCSC Specification Section 4.1. Faying surfaces in snug-tightened joints must meet the requirements in RCSC Specification Sections 3.2 and 3.2.1, but not those for slip-critical joints in RCSC Specification Section 3.2.2. Note that there is generally no need to limit the actual level of pretension provided in snug-tightened joints, per RCSC Specification Section 9.1.

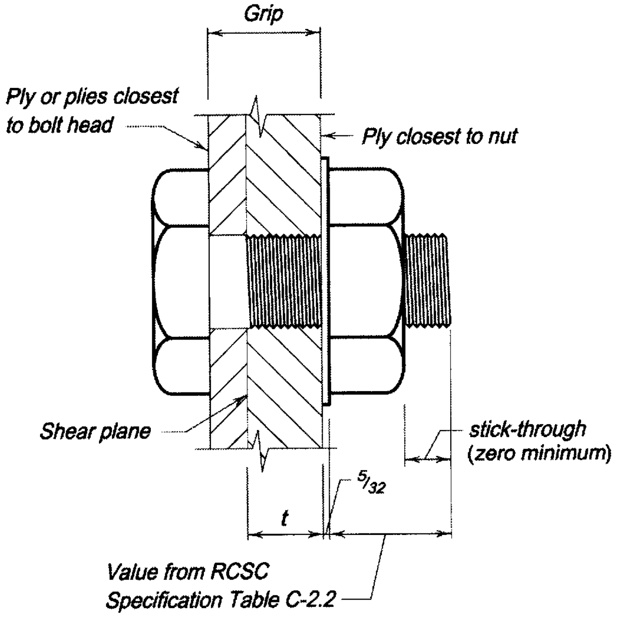


Figure 7–1. Grip and other parameters for bolt length selection.

Pretensioned Joints

When pretension is required but slip-resistance is not of concern, a pretensioned joint should be specified. The applicability is summarized and design requirements, installation requirements, and inspection requirements are stipulated for pretensioned joints per RCSC Specification Section 4.2. Additionally, pretensioned joints are required by default in some cases per AISC Specification Section J1.10. Faying surfaces in pretensioned joints must meet the requirements in RCSC Specification Sections 3.2 and 3.2.1, but not those for slip-critical joints in RCSC Specification Section 3.2.2.

Slip-Critical Joints

The applicability of slip-critical joints is summarized and design requirements, installation requirements, and inspection requirements are stipulated in RCSC Specification Section 4.3. Faying surfaces in slip-critical joints must meet the requirements in RCSC Specification Sections 3.2 and 3.2.2. RCSC defines a faying surface as "the plane of contact between two plies of a joint." Note that the surfaces under the bolt head, washer, and/or nut are not faying surfaces.

Subject to the requirements in RCSC Specification Section 4.3, slip-critical joints are rarely required in building design. Slip-critical joints are appreciably more expensive because of the associated costs of faying-surface preparation. When slip resistance is required and the steel is to be painted, the fabricator should be consulted to determine the most economical approach to providing the necessary slip resistance. Special paint systems that are rated for slip resistance can be specified. Alternatively, a normal paint system can be used with the faying surfaces masked.

DESIGN REQUIREMENTS

Design requirements are found in the AISC Specification as follows. In each case, the available strength determined in accordance with these provisions must equal or exceed the required strength. These requirements are derived from those in the RCSC Specification.

Shear

Available shear strength is determined as given in RCSC Specification Section 5.1 and AISC Specification Section J3.6, with consideration of the presence of fillers and/or shims, per RCSC Specification Section 5.1 and AISC Specification Section J5. When the length of a bolted joint measured parallel to the line of force exceeds 50 in., a 20-percent strength reduction may be applicable, per AISC Specification Table J3.2 footnote f.

Tension

Available tensile strength is determined as given in RCSC Specification Section 5.1 and AISC Specification Section J3.6, with consideration of the effects of prying action, if any. Prying action is a phenomenon (in bolted construction only) whereby the deformation of a fitting under a tensile force increases the tensile force in the bolt. While the effect of prying action is relevant to the design of the bolts, it is primarily a function of the strength and stiffness of the connection elements. Prying action is addressed in Part 9.

Combined Shear and Tension

Available strength for combined shear and tension is determined as given in RCSC Specification Section 5.2 and AISC Specification Section J3.7.

Bearing Strength at Bolt Holes

Available bearing strength at bolt holes is determined as given in RCSC Specification Section 5.3 and AISC Specification Section J3.10.

Slip Resistance

The available strength of slip-critical connections is determined in accordance with AISC Specification Section J3.8. The available strength, ϕR_n or R_n/Ω, is determined by applying the resistance factor or safety factor appropriate for the prevention of slip as a strength or serviceability limit state. In both cases, the required strength is determined using the LRFD load combination for LRFD design and the ASD load combination for ASD design. Slip resistance as a serviceability limit-state is appropriate for most applications.

ECCENTRICALLY LOADED BOLT GROUPS

Eccentricity in the Plane of the Faying Surface

When eccentricity occurs in the plane of the faying surface, the bolts must be designed to resist the combined effect of the direct shear, P_u or P_a, and the additional shear from the induced moment, $P_u e$ or $P_a e$. Two analysis methods for this type of eccentricity are the instantaneous center of rotation method and the elastic method.

The instantaneous center of rotation method is more accurate, but generally requires the use of tabulated values or an iterative solution. The elastic method is simplified, but may be excessively conservative because it neglects the ductility of the bolt group and the potential for load redistribution.

Instantaneous Center of Rotation Method

Eccentricity produces both a rotation and a translation of one connection element with respect to the other. The combined effect of this rotation and translation is equivalent to a rotation about a point defined as the instantaneous center of rotation (IC), as illustrated in Figure 7-2a. The location of the IC depends upon the geometry of the bolt group as well as the direction and point of application of the load.

The load-deformation relationship for one bolt is illustrated in Figure 7-3, where

$$R = R_{ult}(1 - e^{-10\Delta})^{0.55}$$

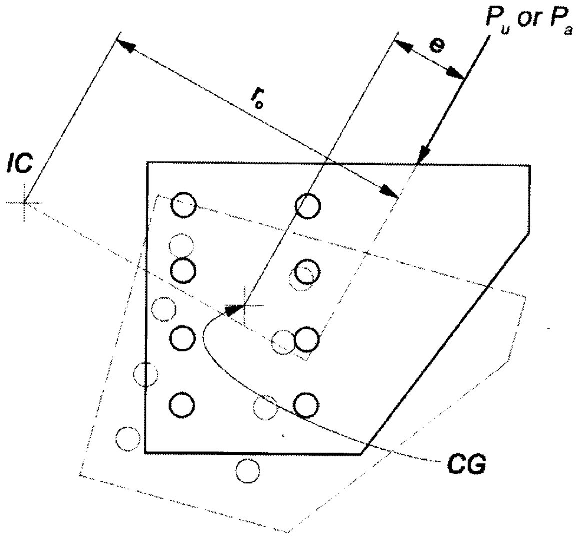
where

R = nominal shear strength of one bolt at a deformation Δ, kips.

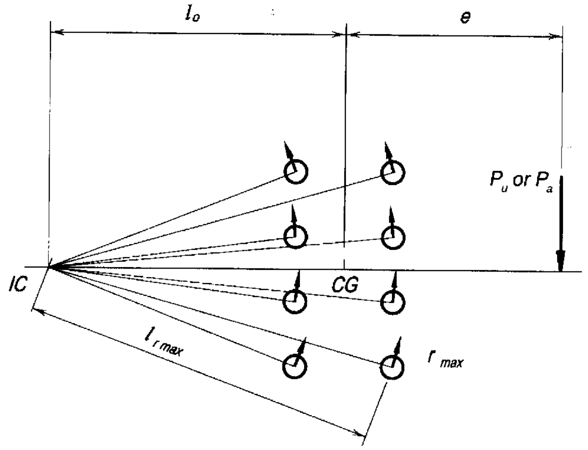
R_{ult} = ultimate shear strength of one bolt, kips.

Δ = total deformation, including shear, bearing and bending deformation in the bolt and bearing deformation of the connection elements, in.

e = 2.718..., base of the natural logarithm.



(a) Instantaneous center of rotation (IC)



(b) Forces on bolts in group for case of θ = 0° for simplicity

Figure 7–2. Illustration for instantaneous center of rotation method.

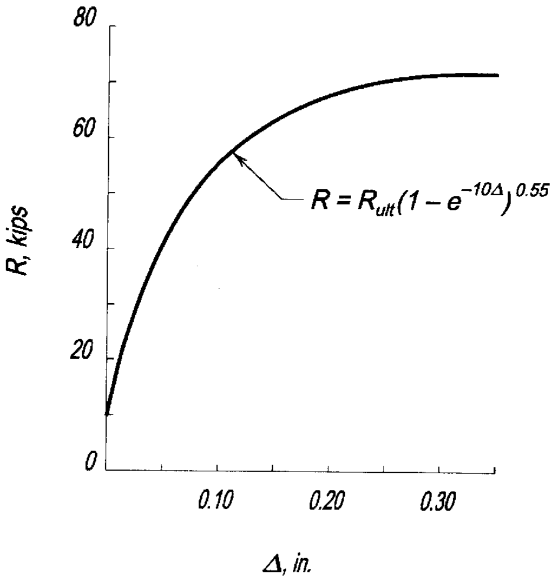


Figure 7–3. Load-definition relationship for one $^3/_4$*-in. diameter ASTM A325 bolt.*

The nominal shear strength of the bolt most remote from the IC can be determined by applying a maximum deformation Δ_{max} to that bolt. The load-deformation relationship is based upon data obtained experimentally for ¾-in. diameter ASTM A325 bolts, where $R_{ult} = 74$ kips, and $\Delta_{max} = 0.34$ in.

The nominal shear strengths of the other bolts in the joint can be determined by applying a deformation Δ that varies linearly with distance from the IC. The nominal shear strength of the bolt group is, then, the sum of the individual strengths of all bolts.

The individual resistance of each bolt is assumed to act on a line perpendicular to a ray passing through the IC and the centroid of that bolt, as illustrated in Figure 7–2b. If the correct location of the IC has been selected, the three equations of in-plane static equilibrium ($\Sigma F_x = 0$, $\Sigma F_y = 0$, and $\Sigma M = 0$) will be satisfied.

For further information, see Crawford and Kulak (1968).

Elastic Method

For a force applied as illustrated in Figure 7–4, the eccentric force, P_u or P_a, is resolved into a direct shear, P_u or P_a, acting through the center of gravity (CG) of the bolt group and a moment, $P_u e$ or $P_a e$, where e is the eccentricity. Each bolt is then assumed to resist an equal share of the direct shear and a share of the eccentric moment proportional to its distance from the CG. The resultant vectorial sum of these forces is the required strength for the bolt, r_u or r_a.

The shear per bolt due to the concentric force, P_u *or* P_a, is r_p, where

LRFD	ASD
$r_p = \frac{P_u}{n}$	$r_p = \frac{P_a}{n}$

and n is the number of bolts. To determine the resultant forces on each bolt when P_u or P_a is applied at an angle θ with respect to vertical, r_p must be resolved into horizontal component, r_{px}, and vertical component, r_{py}, where

$$r_{px} = r_p \sin\theta \text{ and } r_{py} = r_p \cos\theta$$

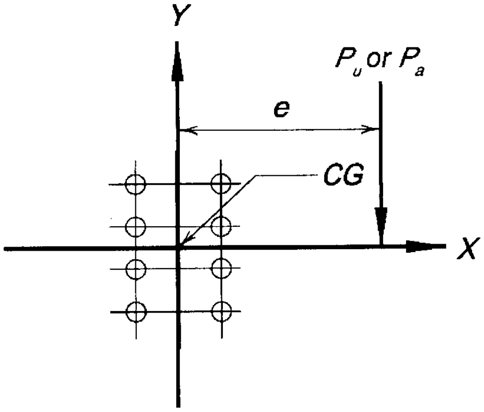


Figure 7–4. Illustration for elastic method.

The shear on the bolt most remote from the CG due to the moment, $P_u e$ or $P_a e$, is r_m, determined as

LRFD	ASD
$r_m = \dfrac{P_u ec}{I_p}$	$r_m = \dfrac{P_a ec}{I_p}$

where

c = radial distance from CG to center of bolt most remote from CG, in.
$I_p = I_x + I_y$ = polar moment of inertia of the bolt group, in.^{4} per in.^{2}

To determine the resultant force on the most highly stressed bolt, r_m must be resolved into horizontal component, r_{mx}, and vertical component, r_{my}, where

LRFD	ASD
$r_{mx} = \dfrac{P_u ec_y}{I_p}$ and $r_{my} = \dfrac{P_u ec_x}{I_p}$	$r_{mx} = \dfrac{P_a ec_y}{I_p}$ and $r_{my} = \dfrac{P_a ec_x}{I_p}$

In the above equation, c_x and c_y are the horizontal and vertical components of the diagonal distance c. Thus, the required strength per bolt is r_u, where

LRFD	ASD
$r_u = \sqrt{\left(r_{px} + r_{mx}\right)^2 + \left(r_{py} + r_{my}\right)^2}$	$r_a = \sqrt{\left(r_{px} + r_{mx}\right)^2 + \left(r_{py} + r_{my}\right)^2}$

For further information, see Higgins (1971).

Eccentricity Normal to the Plane of the Faying Surface

Eccentricity normal to the plane of the faying surface produces tension above and compression below the neutral axis for a bracket connection as shown in Figure 7–5. The eccentric force, P_u *or* P_a, is resolved into a direct shear, P_u *or* P_a, acting at the faying surface of the joint and a moment normal to the plane of the faying surface, $P_u e$ or $P_a e$, where e is the eccentricity. Each bolt is then assumed to resist an equal share of the concentric force, P_u *or* P_a, and the moment is resisted by tension in the bolts above the neutral axis and compression below the neutral axis.

Two design approaches for this type of eccentricity are available: Case I, in which the neutral axis is not taken at the center of gravity (CG), and Case II, in which the neutral axis is taken at the CG.

Case I—Neutral Axis Not at Center of Gravity

The shear per bolt due to the concentric force, r_{uv} or r_{av}, is determined as

LRFD	ASD
$r_{uv} = \dfrac{P_u}{n}$	$r_{av} = \dfrac{P_a}{n}$

where n is the number of bolts in the connection.

A trial position for the neutral axis can be selected at one-sixth of the total bracket depth, measured upward from the bottom (line X-X in Figure 7–6a). To provide for reasonable proportions and to account for the bending stiffness of the connection elements, the effective width of the compression block b_{eff} should be taken as

$$b_{eff} = 8t_f \leq b_f$$

where

t_f = lesser connection element thickness, in.
b_f = connection element width, in.

This effective width is valid for bracket flanges made from W-shapes, S-shapes, welded plates, and angles. Where the bracket flange thickness is not constant, the average flange thickness should be used.

The assumed location of the neutral axis can be evaluated by checking static equilibrium assuming an elastic stress distribution. Equating the moment of the bolt area above the neutral axis with the moment of the compression block area below the neutral axis,

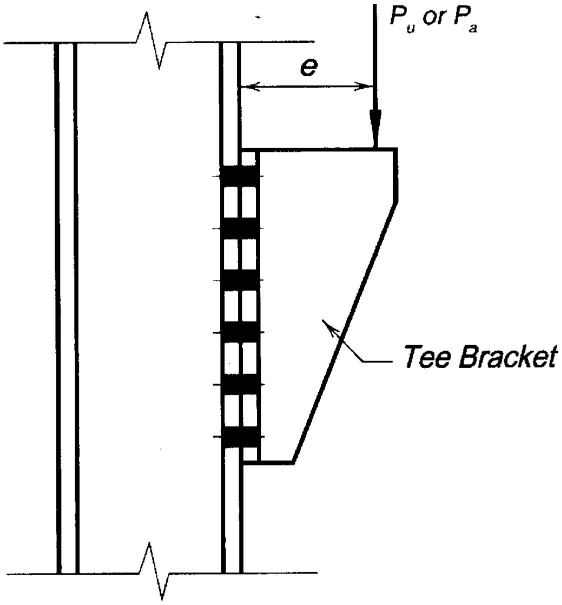


Figure 7–5. Tee bracket subject to eccentric loading normal to the plane of the faying surface.

$$\Sigma A_b \times y = b_{eff} \times d \times d/2$$

In the above equation,

ΣA_b = sum of the areas of all bolts above the neutral axis, in.^{2}
y = distance from line X-X to CG of the bolt group above neutral axis, in.
d = depth of compression block, in.

The value of d may then be adjusted until a reasonable equality exists.

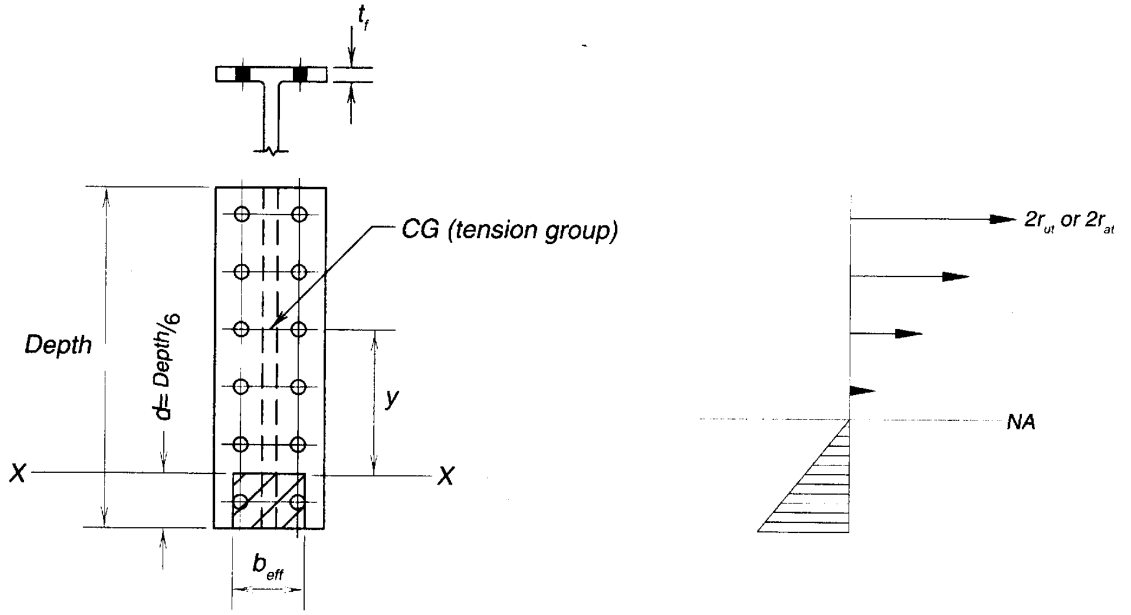
Once the neutral axis has been located, the tensile force per bolt, r_{ut} or r_{at}, as illustrated in Figure 7–6b, may be determined as

LRFD	ASD
$r_{ut} = \dfrac{P_u ec}{I_x} \times A_b$	$r_{at} = \dfrac{P_a ec}{I_x} \times A_b$

where

c = distance from neutral axis to most remote bolt in group, in.
I_x = combined moment of inertia of bolt group and compression block about neutral axis, in.^{4}

Bolts above the neutral axis are subjected to the shear force, the tensile force, and the effect of prying action (see Part 9); bolts below the neutral axis are subjected to the shear force only.



(a) Initial approximation of location of NA

(b) Force diagram with final location of NA

Figure 7–6. Location of neutral axis (NA) for out-of-plane eccentric loading using Case I.

Case II—Neutral Axis at Center of Gravity

This method provides a more direct, but also a more conservative result. As for Case I, the shear force per bolt, r_{uv} or r_{av}, due to the concentric force, P_u or P_a, is determined as

LRFD	ASD
$r_{uv} = \dfrac{P_u}{n}$	$r_{av} = \dfrac{P_a}{n}$

where n is the number of bolts in the connection.

The neutral axis is assumed to be located at the CG of the bolt group as illustrated in Figure 7–7. The bolts above the neutral axis are in tension and the bolts below the neutral axis are said to be in "compression." To obtain a more accurate result, a plastic stress distribution is assumed; this assumption is justified because this method is still more conservative than Case I. Accordingly, the tensile force in each bolt above the neutral axis, r_{ut} or r_{at}, due to the moment, P_ue or P_ae, is determined as

LRFD	ASD
$r_{ut} = \dfrac{P_u e}{n' d_m}$	$r_{at} = \dfrac{P_a e}{n' d_m}$

where

n' = number of bolts above the neutral axis
d_m = moment arm between resultant tensile force and resultant compressive force, in.

Bolts above the neutral axis are subjected to the shear force, the tensile force, and the effect of prying action (see Part 9); bolts below the neutral axis are subjected to the shear force, r_{uv} or r_{av}, only.

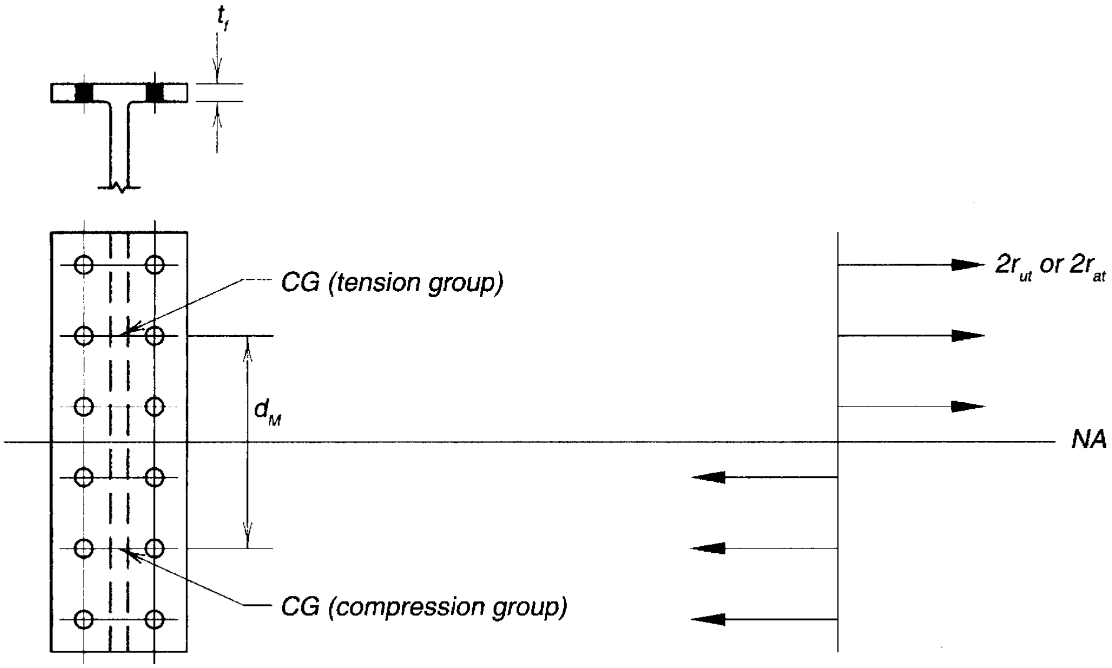


Figure 7–7. Location of neutral axis (NA) for out-of-plane eccentric loading using Case II.

SPECIAL CONSIDERATIONS FOR HOLLOW STRUCTURAL SECTIONS

Through-Bolting to HSS

Long bolts that extend through the entire HSS are satisfactory for shear connections that do not require a fully tensioned installation. The flexibility of the walls of the HSS precludes installation of fully tensioned bolts. Standard structural bolts may be used, although ASTM A449 Bolts may be required for longer lengths. The bolts are designed for static shear and the only limit-state involving the HSS is bolt bearing. The available strength due to shear is determined as ϕR_n or R_n/Ω, with

$$R_n = 1.8nF_y dt$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where

n = number of fasteners
d = fastener diameter, in.
F_y = yield strength of HSS, ksi
t = thickness of HSS, in.

The available strength due to pull-out from tension in one fastener, is determined as ϕR_n or R_n/Ω, with

$$R_n = 0.85d_w tF_u$$

$$\phi = 0.50 \qquad \Omega = 3.00$$

where

d_w = diameter of part in contact with the inner surface of the HSS, in.
F_u = ultimate strength of the HSS, ksi.

Blind Bolts

Special fasteners are available that eliminate the need for access to install a nut (Korol et al, 1993; Henderson, 1996). The shank of the fastener is inserted through holes in the parts to be connected until the head bears on the outer ply (see Figure 7–8). In some cases, a special wrench is used on the open side to keep the outer part of the shank from rotating and simultaneously turn the threaded part of the shank. A wedge or other mechanism on the blind side causes the fixed part of the shank to expand and form a contact with the inside of the HSS. Some fasteners contain a break-off mechanism when the fastener is pretensioned. Recent versions of these fasteners meet the requirements for a pretensioned A325 bolt (Henderson, 1996) and could be used in slip-critical or tension conditions. HSS limit-states are bolt bearing in shear tear-out of the bolt in tension and wall distortion. Manufacturers' literature must be consulted to determine the available strength of blind bolts.

Flow-Drilling

Flow-drilling is a process that can be used to produce a threaded hole in an HSS to permit blind bolting when the inside of the HSS is inaccessible (Sherman, 1995; Henderson, 1996). The process is to force a hole through the HSS with a carbide conical tool rotating at sufficient speed to produce high rapid heating, which softens the material in a local area. The material that is displaced as the tool is forced through the plate forms a truncated hollow cone (bushing) on the inner surface and a small upset on the outer surface. Tools can be obtained with a milling collar so that the material on the outer surface is removed, producing a flat surface allowing parts to be brought in close contact. A cold-formed tap is then used to roll a thread into the hole without any chips or removal of material. The resulting threaded hole has the approximate dimensions and hardness of a heavy hex nut. Shear and tension strengths of A325 bolts can be developed for certain combinations of bolt size and HSS thickness (see Figure 7–9).

Drilling equipment with suitable rotational speed, torque, and thrust is required, but with small sizes and thicknesses, field installation with conventional tools is possible. The bolts are designed with the normal criteria and the HSS limit-states are bolt bearing in shear and distortion of the HSS wall in tension. HSS strength is not affected by the process except for the reduction in area due to the holes.

Threaded Studs to HSS

Threaded studs are available in $^3/_8$-in. to $^7/_8$-in. diameters and can be shop- or field-welded to an HSS with a stud-welding gun. The connection is similar to a bolted connection with an external nut. The strength of the stud in tension or shear is based on manufacturer's recommendations and tests. The HSS limit state is distortion of the wall. When using threaded studs, countersunk holes must be used in the attached element to clear the weld fillet at the base of the stud.

Nailing to HSS

Power-driven nails that are installed with a powder-actuated gun are satisfactory for pure shear connections where the combined thickness of the attachment and the HSS does not exceed $^1/_2$ in. This system was tested as splices between telescoping round HSS loaded with an axial force (Packer, 1996). The shear resistance of the fasteners is taken as the number of nails times the shear strength of a single nail and ignores any secondary contribution from a dimpling effect between the materials. The limit-state for the HSS is shear-bearing. See Packer (1996).

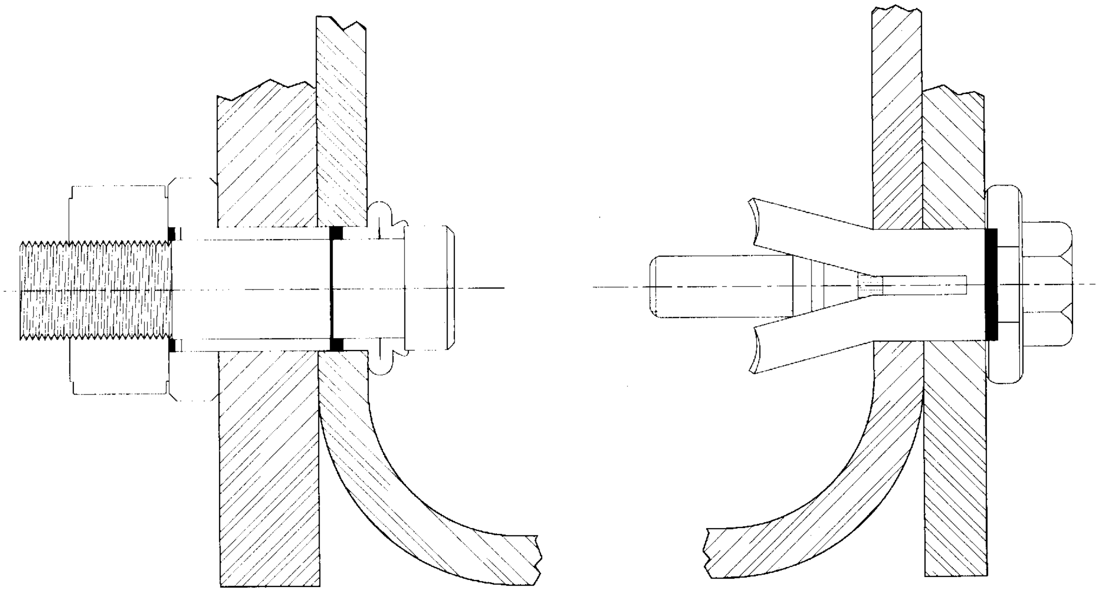


Figure 7–8. Two types of blind bolts.

Screwing To HSS

Self-tapping screws with or without self-drilling points are available for connecting materials with combined thicknesses up to ½ in. The screws have diameters from 0.08 in. to 0.25 in. The limit-states for connections in the AISI *North American Specification for the Design of Cold-Formed Steel Members* (AISI, 2001) are associated with bearing failure of the material or pull-out of the screw either in direct tension or after tilting occurs in a shear load. Failure of the screws themselves is prevented by requiring that the product be 25 percent stronger than the available shear or tension strength of the material. Edge distances and spacing of screws should not exceed 3 times the screw diameter, d. For attaching material with thickness t_1 and ultimate strength F_{u1} to an HSS with thickness t and strength F_u, the available strength, ϕP_n or P_n/Ω, is determined as follows, with $\phi = 0.5$ and $\Omega = 3.0$.

Connection shear per screw

For $t/t_1 \leq 1$, P_n is the smallest of

$$\begin{Bmatrix} 4.2\left(t^3d\right)^{1/2}F_u \\ 2.7tldF_{u1} \\ 2.7tldF_u \end{Bmatrix}$$

For $t/t_1 \geq 2.5$, P_n is the smaller of

$$\begin{Bmatrix} 2.7t_1dF_{u1} \\ 2.7tdF_u \end{Bmatrix}$$

For $1 < t/t_1 < 2.5$, P_n is determined by linear interpolation between the above two cases.

Connection tension per screw, P_n, is the smaller of

$$\begin{Bmatrix} 0.85t_cdF_u \\ 1.5t_1d_wF_{ul} \end{Bmatrix}$$

where

t_c = lesser of the depth of penetration and the HSS thickness
d_w = larger of the screw head or washer diameter, and shall not be taken larger than ½ in.

HSS Thickness (in.)	BOLT DIAMETER (in.)				
	½	⅝	¾	⅞	1
3/16	X	X			
¼	X	X	X		
5/16		X	X	X	
⅜			X	X	X
½					X

Figure 7–9. HSS thickness and bolt diameter combinations.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

The following other specification requirements and design considerations apply to the design of bolts:

Placement of Bolt Groups

For the required placement of bolt groups at the ends of axially loaded members, see AISC Specification Section J1.7.

Bolts in Combination with Welds or Rivets

For bolts used in combination with welds or rivets, see AISC Specification Section J1.8 or J1.9, respectively.

Galvanizing High-Strength Bolts and Nuts

Galvanizing of high-strength bolts is permitted as follows:

1. By the hot-dip or mechanical process for ASTM A325 Type 1 high-strength bolts, per ASTM A325 Section 4.3.
2. By the mechanical process only for ASTM F1852 twist-off-type tension-control bolt assemblies, per ASTM F1852 Section 6.3.
3. By the hot-dip or mechanical process for ASTM A449 bolts, per ASTM A449 Section 5.1.

Nuts for ASTM A325 and F1852 bolts must be galvanized by the same process as the bolt with which they are used. See RCSC Specification Table 2.1 for compatible nut grade and finish requirements for ASTM A325 and F1852 bolts, and ASTM A563 for compatible nut grade and finish requirements for ASTM A449 bolts.

ASTM A490 bolts are not permitted to be galvanized, per ASTM A490 Section 5.4. See also RCSC Specification Commentary Section 2.3.

Reuse of Bolts

The reuse of high-strength bolts is limited, per RCSC Specification Section 2.3.3. See also Bowman and Betancourt (1991).

Fatigue Applications

For applications involving fatigue, see RCSC Specification Sections 4.2, 4.3, and 5.5 and AISC Specification Appendix 3.

Entering and Tightening Clearances

Clearances must be provided for the entering and tightening of the bolts with an impact wrench. The clearance requirements for conventional high-strength bolts are as given in Table 7–16. When high-strength tension-control bolts are specified, the clearance requirements are as given in Table 7–17.

Fully Threaded ASTM A325 Bolts

ASTM A325 bolts with length equal to or less than four times the nominal bolt diameter may be ordered as fully threaded with the designation ASTM A325T. Fully threaded ASTM A325T bolts are not for use in bearing-type X connections since it would be impossible to exclude the threads from the shear plane. While this supplementary provision exists for ASTM A325 bolts, there is no similar supplementary provision made in ASTM A490 for full-length threading.

ASTM A307 Bolts

Limitations are provided on the use of ASTM A307 bolts, per AISC Specification Sections A3.3, J1.8, and J1.10. ASTM A307 bolts are available with both hex and square heads in diameters from $^1/_4$ in. to 4 in. in grade A for general applications and grade B for cast-iron-flanged piping joints. ASTM A563 Grade A nuts are recommended for use with ASTM A307 bolts. Other suitable grades are listed in ASTM A563 Table X1.1.

ASTM A449 Bolts

Limitations are provided on the use of ASTM A449 bolts, per AISC Specification Sections A3.3 and J3.1.

DESIGN TABLES

Tables 7–1. Available Shear Strength of Bolts

The available bolt shear strengths of various grades and sizes of bolts are summarized in Table 7–1.

Table 7–2. Available Tensile Strength of Bolts

The available bolt tensile strengths of various grades and sizes of bolts are summarized in Table 7–2.

Tables 7–3 and 7–4. Available Resistance to Slip

The available slip resistances of various grades and sizes of bolts are summarized in Tables 7–3 and 7–4. In Table 7–3, the available resistance to slip is tabulated for slip to occur at service-level forces. In Table 7–4, the available resistance to slip is tabulated for slip to occur at strength-level forces.

Tables 7–5 and 7–6. Available Bearing Strength at Bolt Holes

The available bearing strength at bolt holes is tabulated for various spacings and edge distances in Tables 7–5 and 7–6, respectively. Note that these tables may be applied to bolts with countersunk heads, by subtracting one-half the depth of the countersink from the material thickness, t. As illustrated in Figure 7–10, this is equivalent to subtracting $d_b/4$ from the material thickness, t.

Tables 7–7 through 7–14. Coefficients *C* for Eccentrically Loaded Bolt Groups

Tables 7–7 through 7–14 employ the instantaneous center of rotation method for the bolt patterns and eccentric conditions indicated, and inclined loads at 0°, 15°, 30°, 45°, 60°, and 75°. The tabulated non-dimensional coefficient, C, represents the number of bolts that are effective in resisting the eccentric shear force.

When Analyzing a Known Bolt Group Geometry

For any of the bolt group geometries shown, the available strength of the eccentrically loaded bolt group, ϕR_n or R_n/Ω, is determined as

$$R_n = C \times r_n$$
$$\phi = 0.75 \qquad \Omega = 2.00$$

In the above equation, r_n is the least nominal strength of one bolt determined from the limit-states of bolt shear strength, bearing strength at bolt holes, and slip resistance (if the connection is to be slip-critical).

When Selecting a Bolt Group

The available strength must be greater than or equal to the required strength, P_u or P_a. Thus, by dividing the required strength, P_u *or* P_a, by the available strength of a single bolt, ϕr_n or r_n/Ω, the minimum coefficient, C, is obtained. The bolt group can then be selected from the table corresponding to the appropriate load angle, at the appropriate eccentricity, e_x, for which the coefficient is of that magnitude or greater.

These tables may be used with any bolt diameter and are conservative when used with ASTM A490 bolts (see Kulak, 1975). Linear interpolation within a given table between adjacent values of e_x is permitted. Available strengths determined with these tables provide a reliability equivalent to that for bolts in joints less than 50 in. long subject to shear produced by a concentric load in either bearing-type or slip-critical connections. Although this

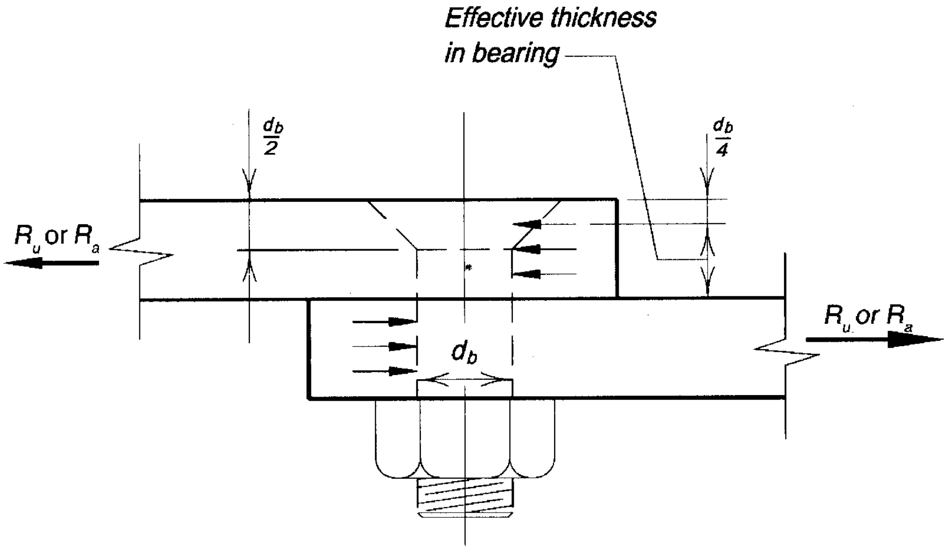


Figure 7–10. Effective bearing-thickness for bolts with countersunk heads.

procedure is based on connections which may experience slip under load, both load tests and analytical studies indicate that it may be conservatively extended to slip-critical connections (Kulak, 1975).

A convergence criterion of one percent was employed for the tabulated iterative solutions. Straight-line interpolation between values for loads at different angles may be significantly unconservative. Either a direct analysis should be performed or the values for the next lower angle increment in the tables should be used for design. For bolt group patterns not treated in these tables, a direct analysis is required if the instantaneous center of rotation method is to be used.

In some cases, it is necessary to calculate the pure moment strength of a bolt group for purposes of linear interpolation. For these cases, the value of C' has been provided for a load angle of 0°. This moment strength of the bolt group is based on the instantaneous center of rotation method and, since a moment-only condition is assumed, the instantaneous center of rotation coincides with the center of gravity of the bolt group. In this case, the strength can be calculated as:

$$M_{max} = 1.25 F_{nv} A_b C'$$

where

$$C' = \sum \left[l_i \left(1 - e^{-\left(\frac{10 l_i \Delta_{max}}{l_{max}} \right)} \right)^{0.55} \right]$$

F_{nv} = the shear strength of an individual bolt from AISC Specification Table J3.2
l_i = the distance of the i^{th} bolt from the center of gravity of the bolt group
Δ_{max} = the maximum deformation on the bolt furthest from the center of gravity, 0.34 in.
l_{max} = the distance of the bolt farthest from the center of gravity of the bolt group to the center

Table 7–15. Dimensions of High-Strength Fasteners

Dimensions of ASTM A325 and A490 bolts, A563 nuts, and F436 washers are given and illustrated in Table 7–15.

Table 7–16 and 17. Entering and Tightening Clearances

Clearance is required for entering and tightening bolts with an impact wrench. The required clearances are given for conventional high-strength bolts and twist-off-type tension-control bolt assemblies in Tables 7–16 and 7–17, respectively.

Table 7–18. Threading Dimensions for High-Strength and Non-High-Strength Bolts

Data regarding the characteristics of the threading dimensions of high-strength and non-high-strength bolts is provided in Table 7–18.

Table 7–19. Weights of High-Strength Fasteners

Weights of conventional ASTM A325 and A490 bolts, A563 nuts, and F436 washers are given in Table 7–19. For dimensions and weights of tension-control ASTM A325 and A490 bolts, refer to manufacturers' literature or the Industrial Fasteners Institute (IFI). For dimensions and weights of ASTM A449 bolts, refer to Table 7–20.

Table 7–20. Dimensions of Non-High-Strength Fasteners

Typical non-high-strength bolt head and nut dimensions are given in Table 7–21. Thread lengths listed in this table may be calculated for non-high-strength bolts as $2d_b + \frac{1}{4}$ in. for bolts up to 6 in. long and $2d_b + \frac{1}{2}$ in. for bolts over 6 in. long, where d_b is the bolt diameter. Note that these thread lengths are longer than those given previously for high-strength bolts in Table 7–15. Threading dimensions are given in Table 7–18.

Tables 7–21, 7–22, and 7–23. Weights of Non-High-Strength Fasteners

Weights of non-high-strength bolts are given in Tables 7–21, 7–22, and 7–23.

PART 7 REFERENCES

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Table 7-1
Available Shear Strength of Bolts, kips

Nominal Bolt Diameter d_b, in.					$^5/_8$		$^3/_4$		$^7/_8$		1	
Nominal Bolt Area, in.^{2}					0.307		0.442		0.601		0.785	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 F1852	N	24.0	36.0	S	7.36	11.0	10.6	15.9	14.4	21.6	18.8	28.3
				D	14.7	22.1	21.2	31.8	28.9	43.3	37.7	56.5
	X	30.0	45.0	S	9.20	13.8	13.3	19.9	18.0	27.1	23.6	35.3
				D	18.4	27.6	26.5	39.8	36.1	54.1	47.1	70.7
A490	N	30.0	45.0	S	9.20	13.8	13.3	19.9	18.0	27.1	23.6	35.3
				D	18.4	27.6	26.5	39.8	36.1	54.1	47.1	70.7
	X	37.5	56.3	S	11.5	17.3	16.6	24.9	22.5	33.8	29.5	44.2
				D	23.0	34.5	33.1	49.7	45.1	67.6	58.9	88.4
A307	–	12.0	18.0	S	3.68	5.52	5.30	7.95	7.22	10.8	9.42	14.1
				D	7.36	11.0	10.6	15.9	14.4	21.6	18.8	28.3

Nominal Bolt Diameter d_b, in.					$1^1/_8$		$1^1/_4$		$1^3/_8$		$1^1/_2$	
Nominal Bolt Area, in.^{2}					0.994		1.23		1.48		1.77	
ASTM Desig.	Thread Cond.	F_{nv}/Ω (ksi)	ϕF_{nv} (ksi)	Loading	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	LRFD		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 F1852	N	24.0	36.0	S	23.9	35.8	29.5	44.2	35.6	53.5	42.4	63.6
				D	47.7	71.6	58.9	88.4	71.3	107	84.8	127
	X	30.0	45.0	S	29.8	44.7	36.8	55.2	44.5	66.8	53.0	79.5
				D	59.6	89.5	73.6	110	89.1	134	106	159
A490	N	30.0	45.0	S	29.8	44.7	36.8	55.2	44.5	66.8	53.0	79.5
				D	59.6	89.5	73.6	110	89.1	134	106	159
	X	37.5	56.3	S	37.3	55.9	46.0	69.0	55.7	83.5	66.3	99.4
				D	74.6	112	92.0	138	111	167	133	199
A307	–	12.0	18.0	S	11.9	17.9	14.7	22.1	17.8	26.7	21.2	31.8
				D	23.9	35.8	29.5	44.2	35.6	53.5	42.4	63.6

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

Table 7–2
Available Tensile Strength of Bolts, kips

Nominal Bolt Diameter d_b, in.			5/8		3/4		7/8		1	
Nominal Bolt Area, in.^{2}			0.307		0.442		0.601		0.785	
ASTM Desig.	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 & F1852	45.0	67.5	13.8	20.7	19.9	29.8	27.1	40.6	35.3	53.0
A490	56.5	84.8	17.3	26.0	25.0	37.4	34.0	51.0	44.4	66.6
A307	22.5	33.8	6.90	10.4	9.94	14.9	13.5	20.3	17.7	26.5

Nominal Bolt Diameter d_b, in.			1 1/8		1 1/4		1 3/8		1 1/2	
Nominal Bolt Area, in.^{2}			0.994		1.23		1.48		1.77	
ASTM Desig.	F_{nt}/Ω (ksi)	ϕF_{nt} (ksi)	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n	r_n/Ω	ϕr_n
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
A325 & F1852	45.0	67.5	44.7	67.1	55.2	82.8	66.8	100	79.5	119
A490	56.5	84.8	56.2	84.2	69.3	104	83.9	126	99.8	150
A307	22.5	33.8	22.4	33.5	27.6	41.4	33.4	50.1	39.8	59.6

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

A325

Table 7–3

Slip-Critical Connections

Available Shear Strength, kips, when

Slip is a Serviceability Limit-State

(Class A Faying Surface, $\mu = 0.35$)

ASTM A325 / F1852 Bolts

Hole Type	Loading	Nominal Bolt Diameter d, in.							
		$^5/_8$		$^3/_4$		$^7/_8$		1	
		Minimum ASTM A325/F1852 Bolt Pretension, kips							
		19		28		39		51	
		r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
STD	**S**	5.01	7.51	7.38	11.1	10.3	15.4	13.4	20.2
	D	10.0	15.0	14.8	22.1	20.6	30.8	26.9	40.3
OVS/SSL	**S**	4.26	6.39	6.28	9.41	8.74	13.1	11.4	17.1
	D	8.52	12.8	12.6	18.8	17.5	26.2	22.9	34.3
LSL	**S**	3.51	5.26	5.17	7.75	7.20	10.8	9.41	14.1
	D	7.01	10.5	10.3	15.5	14.4	21.6	18.8	28.2

Hole Type	Loading	Nominal Bolt Diameter d, in.							
		$1^1/_8$		$1^1/_4$		$1^3/_8$		$1^1/_2$	
		Minimum ASTM A325/F1852 Bolt Pretension, kips							
		56		71		85		103	
		r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
STD	**S**	14.8	22.1	18.7	28.1	22.4	33.6	27.2	40.7
	D	29.5	44.3	37.4	56.2	44.8	67.2	54.3	81.5
OVS/SSL	**S**	12.6	18.8	15.9	23.9	19.0	28.6	23.1	34.6
	D	25.1	37.7	31.8	47.7	38.1	57.1	46.2	69.3
LSL	**S**	10.3	15.5	13.1	19.7	15.7	23.5	19.0	28.5
	D	20.7	31.0	26.2	39.3	31.4	47.1	38.0	57.0

STD = Standard Hole
SSL = Short-Slotted Hole
LSL = Long-Slotted Hole
OVS = Oversized Hole

S = Single Shear
D = Double Shear

ASD	LRFD
$\Omega_v = 1.50$	$\phi_v = 1.00$

Note: For available slip resistance when slip is a strength limit state, see Table 7-4.
For Class B faying surfaces ($\mu = 0.50$), multiply the tabulated available strength by 1.43.
The required strength is determined using LRFD load combinations for LRFD design and ASD load combinations for ASD design.

Table 7-3 (continued)
Slip-Critical Connections
Available Shear Strength, kips, when
Slip is a Serviceability Limit-State
(Class A Faying Surface, $\mu = 0.35$)

A490

ASTM A490 Bolts

Hole Type	Loading	Nominal Bolt Diameter *d*, in.							
		$\frac{5}{8}$		$\frac{3}{4}$		$\frac{7}{8}$		1	
		Minimum ASTM A490 Bolt Pretension, kips							
		24		35		49		64	
		r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD	S	6.33	9.49	9.23	13.8	12.9	19.4	16.9	25.3
	D	12.7	19.0	18.5	27.7	25.8	38.8	33.7	50.6
OVS/SSL	S	5.38	8.07	7.84	11.8	11.0	16.5	14.3	21.5
	D	10.8	16.1	15.7	23.5	22.0	32.9	28.7	43.0
LSL	S	4.43	6.64	6.46	9.69	9.04	13.6	11.8	17.7
	D	8.86	13.3	12.9	19.4	18.1	27.1	23.6	35.4

Hole Type	Loading	Nominal Bolt Diameter *d*, in.							
		$1\frac{1}{8}$		$1\frac{1}{4}$		$1\frac{3}{8}$		$1\frac{1}{2}$	
		Minimum ASTM A490 Bolt Pretension, kips							
		80		102		121		148	
		r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD	S	21.1	31.6	26.9	40.3	31.9	47.9	39.0	58.5
	D	42.2	63.3	53.8	80.7	63.8	95.7	78.0	117
OVS/SSL	S	17.9	26.9	22.9	34.3	27.1	40.7	33.2	49.8
	D	35.9	53.8	45.7	68.6	54.2	81.4	66.3	99.5
LSL	S	14.8	22.1	18.8	28.2	22.3	33.5	27.3	41.0
	D	29.5	44.3	37.7	56.5	44.7	67.0	54.6	81.9

STD = Standard Hole
SSL = Short-Slotted Hole
LSL = Long-Slotted Hole
OVS = Oversized Hole

S = Single Shear
D = Double Shear

ASD	LRFD	
$\Omega_v = 1.50$	$\phi_v = 1.00$	Note: For available slip resistance when slip is a strength limit state, see Table 7-4. For Class B faying surfaces (μ = 0.50), multiply the tabulated available strength by 1.43. The required strength is determined using LRFD load combinations for LRFD design and ASD load combinations for ASD design.

A325

Table 7–4
Slip-Critical Connections
Available Shear Strength, kips, when
Slip is a Strength Limit-State
(Class A Faying Surface, μ = 0.35)

ASTM A325 / F1852 Bolts

Hole Type	Loading	Nominal Bolt Diameter *d*, in.							
		5/8		3/4		7/8		1	
		Minimum ASTM A325/F1852 Bolt Pretension, kips							
		19		28		39		51	
		r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
STD	**S**	4.29	6.39	6.33	9.41	8.81	13.1	11.5	17.1
	D	8.59	12.8	12.7	18.8	17.6	26.2	23.1	34.3
OVS/SSL	**S**	3.65	5.43	5.38	8.00	7.49	11.1	9.80	14.6
	D	7.30	10.9	10.8	16.0	15.0	22.3	19.6	29.1
LSL	**S**	3.01	4.47	4.43	6.59	6.17	9.18	8.07	12.0
	D	6.01	8.94	8.86	13.2	12.3	18.4	16.1	24.0

Hole Type	Loading	Nominal Bolt Diameter *d*, in.							
		1 1/8		1 1/4		1 3/8		1 1/2	
		Minimum ASTM A325/F1852 Bolt Pretension, kips							
		56		71		85		103	
		r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
STD	**S**	12.7	18.8	16.0	23.9	19.2	28.6	23.3	34.6
	D	25.3	37.7	32.1	47.7	38.4	57.1	48.6	69.3
OVS/SSL	**S**	10.8	16.0	13.6	20.3	16.3	24.3	19.8	29.4
	D	21.5	32.0	27.3	40.6	32.7	48.6	39.6	58.6
LSL	**S**	8.86	13.2	11.2	16.7	13.4	20.0	16.3	24.2
	D	17.7	26.4	22.5	33.4	26.9	40.0	32.6	48.5

STD = Standard Hole
SSL = Short-Slotted Hole
LSL = Long-Slotted Hole
OVS = Oversized Hole

S = Single Shear
D = Double Shear

ASD	LRFD	
$\Omega_v = 1.76$	$\phi_v = 0.85$	Note: For available slip resistance when slip is a serviceability limit state, see Table 7-3. For Class B faying surfaces (μ = 0.50), multiply the tabulated available strength by 1.43. The required strength is determined using LRFD load combinations for LRFD design and ASD load combinations for ASD design.

Table 7–4 (continued)
Slip-Critical Connections
Available Shear Strength, kips, when
Slip is a Strength Limit-State
(Class A Faying Surface, μ = 0.35)

A490

ASTM A490 Bolts

Hole Type	Loading	Nominal Bolt Diameter d, in.							
		5/8		3/4		7/8		1	
		Minimum ASTM A490 Bolt Pretension, kips							
		24		35		49		64	
		r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD	S	5.42	8.07	7.91	11.8	11.1	16.5	14.5	21.5
	D	10.8	16.1	15.8	23.5	22.1	32.9	28.9	43.0
OVS/SSL	S	4.61	6.86	6.72	10.0	9.41	14.0	12.3	18.3
	D	9.22	13.7	13.4	20.0	18.8	28.0	24.6	36.6
LSL	S	3.80	5.65	5.54	8.24	7.75	11.5	10.0	15.1
	D	7.59	11.3	11.1	16.5	15.5	23.1	20.2	30.1

Hole Type	Loading	Nominal Bolt Diameter d, in.							
		1 1/8		1 1/4		1 3/8		1 1/2	
		Minimum ASTM A490 Bolt Pretension, kips							
		80		102		121		148	
		r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD	S	18.1	26.9	23.1	34.3	27.3	40.7	33.4	49.8
	D	36.2	53.8	46.1	68.6	54.7	81.4	66.9	99.5
OVS/SSL	S	15.4	22.9	19.6	29.1	23.2	34.6	28.4	42.3
	D	30.7	45.7	39.2	58.3	46.5	69.2	56.9	84.6
LSL	S	12.7	18.8	16.1	24.0	19.1	28.5	23.4	34.8
	D	25.3	37.7	32.3	48.0	38.3	56.9	46.8	69.7

STD = Standard Hole
SSL = Short-Slotted Hole
LSL = Long-Slotted Hole
OVS = Oversized Hole

S = Single Shear
D = Double Shear

ASD	LRFD
$\Omega_v = 1.76$	$\phi_v = 0.85$

Note: For available slip resistance when slip is a serviceability limit state, see Table 7-3.
For Class B faying surfaces (μ = 0.50), multiply the tabulated available strength by 1.43.
The required strength is determined using LRFD load combinations for LRFD design and ASD load combinations for ASD design.

Table 7–5

Available Bearing Strength at Bolt Holes Based on Bolt Spacing

kips/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	Nominal Bolt Diameter d_b, in.							
			$^5/_8$		$^3/_4$		$^7/_8$		1	
			r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	$2^2/_3\,d_b$	58	34.1	51.1	41.3	62.0	48.6	72.9	55.8	83.7
		65	38.2	57.3	46.3	69.5	54.4	81.7	62.6	93.8
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	67.4	101
		65	48.8	73.1	58.5	87.8	68.3	102	75.6	113
SSLP	$2^2/_3\,d_b$	58	27.6	41.3	34.8	52.2	42.1	63.1	47.1	70.7
		65	30.9	46.3	39.0	58.5	47.1	70.7	52.8	79.2
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	58.7	88.1
		65	48.8	73.1	58.5	87.8	68.3	102	65.8	98.7
OVS	$2^2/_3\,d_b$	58	29.7	44.6	37.0	55.5	44.2	66.3	49.3	74.0
		65	33.3	50.0	41.4	62.2	49.6	74.3	55.3	82.9
	3 in.	58	43.5	65.3	52.2	78.3	60.9	91.4	60.9	91.4
		65	48.8	73.1	58.5	87.8	68.3	102	68.3	102
LSLP	$2^2/_3\,d_b$	58	3.62	5.44	4.35	6.53	5.08	7.61	5.80	8.70
		65	4.06	6.09	4.88	7.31	5.69	8.53	6.50	9.75
	3 in.	58	43.5	65.3	39.2	58.7	28.3	42.4	17.4	26.1
		65	48.8	73.1	43.9	65.8	31.7	47.5	19.5	29.3
LSLT	$2^2/_3\,d_b$	58	28.4	42.6	34.4	51.7	40.5	60.7	46.5	69.8
		65	31.8	47.7	38.6	57.9	45.4	68.0	52.1	78.2
	3 in.	58	36.3	54.4	43.5	65.3	50.8	76.1	56.2	84.3
		65	40.6	60.9	48.8	73.1	56.9	85.3	63.0	94.5
STD, SSLT, SSLP, OVS, LSLP	$s \geq s_{full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$s \geq s_{full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Spacing for full bearing strength s_{full}[a], in.		STD, SSLT, LSLT	$1^{15}/_{16}$		$2^5/_{16}$		$2^{11}/_{16}$		$3^1/_{16}$	
		OVS	$2^1/_{16}$		$2^7/_{16}$		$2^{13}/_{16}$		$3^1/_4$	
		SSLP	$2^1/_8$		$2^1/_2$		$2^7/_8$		$3^5/_{16}$	
		LSLP	$2^{13}/_{16}$		$3^3/_8$		$3^{15}/_{16}$		$4^1/_2$	
Minimum Spacing[a] = $2^2/_3\,d_b$, in.			$1^{11}/_{16}$		2		$2^5/_{16}$		$2^{11}/_{16}$	

STD = Standard Hole
SSLT = Short-Slotted Hole oriented transverse to the line of force
SSLP = Short-Slotted Hole oriented parallel to the line of force
OVS = Oversized Hole
LSLP = Long-Slotted Hole oriented parallel to the line of force
LSLT = Long-Slotted Hole oriented transverse to the line of force

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.

[a] Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7–5 (continued)
Available Bearing Strength at Bolt Holes Based on Bolt Spacing
kips/in. thickness

Hole Type	Bolt Spacing, s, in.	F_u, ksi	Nominal Bolt Diameter d_b, in.							
			$1\frac{1}{8}$		$1\frac{1}{4}$		$1\frac{3}{8}$		$1\frac{1}{2}$	
			r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	$2\frac{2}{3}\,d_b$	58	63.1	94.6	70.3	105	77.6	116	84.8	127
		65	70.7	106	78.8	118	86.9	130	95.1	143
	3 in.	58	63.1	94.6	—	—	—	—	—	—
		65	70.7	106	—	—	—	—	—	—
SSLP	$2\frac{2}{3}\,d_b$	58	52.2	78.3	59.5	89.2	66.7	100	74.0	111
		65	58.5	87.8	66.6	99.9	74.8	112	82.9	124
	3 in.	58	52.2	78.3	—	—	—	—	—	—
		65	58.5	87.8	—	—	—	—	—	—
OVS	$2\frac{2}{3}\,d_b$	58	54.4	81.6	61.6	92.4	68.9	103	76.1	114
		65	60.9	91.4	69.1	104	77.2	116	85.3	128
	3 in.	58	54.4	81.6	—	—	—	—	—	—
		65	60.9	91.4	—	—	—	—	—	—
LSLP	$2\frac{2}{3}\,d_b$	58	6.53	9.79	7.25	10.9	7.98	12.0	8.70	13.1
		65	7.31	11.0	8.13	12.2	8.94	13.4	9.75	14.6
	3 in.	58	6.53	9.79	—	—	—	—	—	—
		65	7.31	11.0	—	—	—	—	—	—
LSLT	$2\frac{2}{3}\,d_b$	58	52.6	78.8	58.6	87.9	64.6	97.0	70.7	106
		65	58.9	88.4	65.7	98.5	72.4	109	79.2	119
	3 in.	58	52.6	78.8	—	—	—	—	—	—
		65	58.9	88.4	—	—	—	—	—	—
STD, SSLT, SSLP, OVS, LSLP	$s \geq s_{full}$	58	78.3	117	87.0	131	95.7	144	104	157
		65	87.8	132	97.5	146	107	161	117	176
LSLT	$s \geq s_{full}$	58	65.3	97.9	72.5	109	79.8	120	87.0	131
		65	73.1	110	81.3	122	89.4	134	97.5	146
Spacing for full bearing strength s_{full}[a], in.		STD, SSLT, LSLT	$3\frac{7}{16}$		$3\frac{13}{16}$		$4\frac{3}{16}$		$4\frac{9}{16}$	
		OVS	$3\frac{11}{16}$		$4\frac{1}{16}$		$4\frac{7}{16}$		$4\frac{13}{16}$	
		SSLP	$3\frac{3}{4}$		$4\frac{1}{8}$		$4\frac{1}{2}$		$4\frac{7}{8}$	
		LSLP	$5\frac{1}{16}$		$5\frac{5}{8}$		$6\frac{3}{16}$		$6\frac{3}{4}$	
Minimum Spacing[a] = $2\frac{2}{3}\,d_b$, in.			3		$3\frac{5}{16}$		$3\frac{11}{16}$		4	

STD = Standard Hole
SSLT = Short-Slotted Hole oriented transverse to the line of force
SSLP = Short-Slotted Hole oriented parallel to the line of force
OVS = Oversized Hole
LSLP = Long-Slotted Hole oriented parallel to the line of force
LSLT = Long-Slotted Hole oriented transverse to the line of force

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

— indicates spacing less than minimum spacing required per AISC Specification Section J3.3.

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.

[a] Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7–6
Available Bearing Strength at Bolt Holes Based on Edge Distance
kips/in. thickness

Hole Type	Edge Distance L_e, in.	F_u, ksi	Nominal Bolt Diameter d_b, in.							
			5/8		3/4		7/8		1	
			r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$	r_n/Ω_v	$\phi_v r_n$
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
STD SSLT	1 1/4	58	31.5	47.3	29.4	44.0	27.2	40.8	25.0	37.5
		65	35.3	53.0	32.9	49.4	30.5	45.7	28.0	42.0
	2	58	43.5	65.3	52.2	78.3	53.3	79.9	51.1	76.7
		65	48.8	73.1	58.5	87.8	59.7	89.6	57.3	85.9
SSLP	1 1/4	58	28.3	42.4	26.1	39.2	23.9	35.9	20.7	31.0
		65	31.7	47.5	29.3	43.9	26.8	40.2	23.2	34.7
	2	58	43.5	65.3	52.2	78.3	50.0	75.0	46.8	70.1
		65	48.8	73.1	58.5	87.8	56.1	84.1	52.4	78.6
OVS	1 1/4	58	29.4	44.0	27.2	40.8	25.0	37.5	21.8	32.6
		65	32.9	49.4	30.5	45.7	28.0	42.0	24.4	36.6
	2	58	43.5	65.3	52.2	78.3	51.1	76.7	47.9	71.8
		65	48.8	73.1	58.5	87.8	57.3	85.9	53.6	80.4
LSLP	1 1/4	58	16.3	24.5	10.9	16.3	5.44	8.16	—	—
		65	18.3	27.4	12.2	18.3	6.09	9.14	—	—
	2	58	42.4	63.6	37.0	55.5	31.5	47.3	26.1	39.2
		65	47.5	71.3	41.4	62.2	35.3	53.0	29.3	43.9
LSLT	1 1/4	58	26.3	39.4	24.5	36.7	22.7	34.0	20.8	31.3
		65	29.5	44.2	27.4	41.1	25.4	38.1	23.4	35.0
	2	58	36.3	54.4	43.5	65.3	44.4	66.6	42.6	63.9
		65	40.6	60.9	48.8	73.1	49.8	74.6	47.7	71.6
STD, SSLT, SSLP, OVS, LSLP	$L_e \geq L_{e\,full}$	58	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104
		65	48.8	73.1	58.5	87.8	68.3	102	78.0	117
LSLT	$L_e \geq L_{e\,full}$	58	36.3	54.4	43.5	65.3	50.8	76.1	58.0	87.0
		65	40.6	60.9	48.8	73.1	56.9	85.3	65.0	97.5
Edge distance for full bearing strength $L_e \geq L_{e\,full}$[a], *in.*		STD, SSLT, LSLT	1 5/8		1 15/16		2 1/4		2 9/16	
		OVS	1 11/16		2		2 5/16		2 5/8	
		SSLP	1 11/16		2		2 5/16		2 11/16	
		LSLP	2 1/16		2 7/16		2 7/8		3 1/4	

STD = Standard Hole
SSLT = Short-Slotted Hole oriented transverse to the line of force
SSLP = Short-Slotted Hole oriented parallel to the line of force
OVS = Oversized Hole
LSLP = Long-Slotted Hole oriented parallel to the line of force
LSLT = Long-Slotted Hole oriented transverse to the line of force

ASD	LRFD	
$\Omega_v = 2.00$	$\phi_v = 0.75$	— indicates spacing less than minimum spacing required per AISC Specification Section J3.3. Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10. [a] Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7–6 (continued)

Available Bearing Strength at Bolt Holes Based on Edge Distance

kips/in. thickness

Hole Type	Edge Distance L_e, in.	F_u, ksi	Nominal Bolt Diameter d_b, in. $1\frac{1}{8}$ r_n/Ω_v ASD	$1\frac{1}{8}$ $\phi_v r_n$ LRFD	$1\frac{1}{4}$ r_n/Ω_v ASD	$1\frac{1}{4}$ $\phi_v r_n$ LRFD	$1\frac{3}{8}$ r_n/Ω_v ASD	$1\frac{3}{8}$ $\phi_v r_n$ LRFD	$1\frac{1}{2}$ r_n/Ω_v ASD	$1\frac{1}{2}$ $\phi_v r_n$ LRFD
STD SSLT	$1\frac{1}{4}$	58	22.8	34.3	20.7	31.0	18.5	27.7	16.3	24.5
		65	25.6	38.4	23.2	34.7	20.7	31.1	18.3	27.4
	2	58	48.9	73.4	46.8	70.1	44.6	66.9	42.4	63.6
		65	54.8	82.3	52.4	78.6	50.0	75.0	47.5	71.3
SSLP	$1\frac{1}{4}$	58	17.4	26.1	15.2	22.8	13.1	19.6	10.9	16.3
		65	19.5	29.3	17.1	25.6	14.6	21.9	12.2	18.3
	2	58	43.5	65.3	41.3	62.0	39.2	58.7	37.0	55.5
		65	48.8	73.1	46.3	69.5	43.9	65.8	41.4	62.2
OVS	$1\frac{1}{4}$	58	18.5	27.7	16.3	24.5	14.1	21.2	12.0	17.9
		65	20.7	31.1	18.3	27.4	15.8	23.8	13.4	20.1
	2	58	44.6	66.9	42.4	63.6	40.2	60.4	38.1	57.1
		65	50.0	75.0	47.5	71.3	45.1	67.6	42.7	64.0
LSLP	$1\frac{1}{4}$	58	—	—	—	—	—	—	—	—
		65	—	—	—	—	—	—	—	—
	2	58	20.7	31.0	15.2	22.8	9.79	14.7	4.35	6.53
		65	23.2	34.7	17.1	25.6	11.0	16.5	4.88	7.31
LSLT	$1\frac{1}{4}$	58	19.0	28.5	17.2	25.8	15.4	23.1	13.6	20.4
		65	21.3	32.0	19.3	28.9	17.3	25.9	15.2	22.9
	2	58	40.8	61.2	39.0	58.5	37.2	55.7	35.3	53.0
		65	45.7	68.6	43.7	65.5	41.6	62.5	39.6	59.4
STD, SSLT, SSLP, OVS, LSLP	$L_e \geq L_{e\,full}$	58	78.3	117	87.0	131	95.7	144	104	157
		65	87.8	132	97.5	146	107	161	117	176
LSLT	$L_e \geq L_{e\,full}$	58	65.3	97.9	72.5	109	79.8	120	87.0	131
		65	73.1	110	81.3	122	89.4	134	97.5	146
Edge distance for full bearing strength $L_e \geq L_{e\,full}$[a], in.		STD, SSLT, LSLT	$2\frac{7}{8}$		$3\frac{3}{16}$		$3\frac{1}{2}$		$3\frac{13}{16}$	
		OVS	3		$3\frac{5}{16}$		$3\frac{5}{8}$		$3\frac{15}{16}$	
		SSLP	3		$3\frac{5}{16}$		$3\frac{5}{8}$		$3\frac{15}{16}$	
		LSLP	$3\frac{11}{16}$		$4\frac{1}{16}$		$4\frac{1}{2}$		$4\frac{7}{8}$	

STD = Standard Hole
SSLT = Short-Slotted Hole oriented transverse to the line of force
SSLP = Short-Slotted Hole oriented parallel to the line of force
OVS = Oversized Hole
LSLP = Long-Slotted Hole oriented parallel to the line of force
LSLT = Long-Slotted Hole oriented transverse to the line of force

ASD	LRFD
$\Omega_v = 2.00$	$\phi_v = 0.75$

— indicates spacing less than minimum spacing required per AISC Specification Section J3.3.

Note: Spacing indicated is from the center of the hole or slot to the center of the adjacent hole or slot in the line of force. Hole deformation is considered. When hole deformation is not considered, see AISC Specification Section J3.10.

[a] Decimal value has been rounded to the nearest sixteenth of an inch.

Table 7-7
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

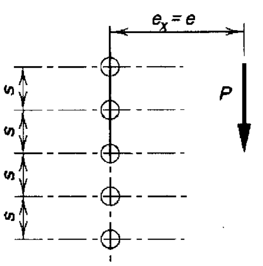
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
3	2	1.18	2.23	3.32	4.39	5.45	6.48	7.51	8.52	9.53	10.5	11.5
	3	0.88	1.75	2.81	3.90	4.98	6.06	7.12	8.17	9.21	10.2	11.3
	4	0.69	1.40	2.36	3.40	4.47	5.56	6.64	7.72	8.78	9.84	10.9
	5	0.56	1.15	2.01	2.96	3.98	5.05	6.13	7.22	8.30	9.38	10.4
	6	0.48	0.97	1.73	2.59	3.55	4.57	5.63	6.70	7.79	8.87	9.96
	7	0.41	0.83	1.51	2.28	3.17	4.13	5.15	6.20	7.28	8.36	9.44
	8	0.36	0.73	1.34	2.04	2.85	3.75	4.72	5.73	6.78	7.85	8.93
	9	0.32	0.65	1.21	1.83	2.59	3.42	4.34	5.31	6.32	7.36	8.42
	10	0.29	0.59	1.09	1.66	2.36	3.14	4.00	4.92	5.89	6.90	7.94
	12	0.24	0.49	0.92	1.40	2.00	2.68	3.44	4.27	5.15	6.09	7.06
	14	0.21	0.42	0.79	1.21	1.74	2.33	3.01	3.75	4.55	5.41	6.31
	16	0.18	0.37	0.70	1.06	1.53	2.06	2.67	3.33	4.06	4.85	5.68
	18	0.16	0.33	0.62	0.95	1.37	1.84	2.39	3.00	3.66	4.38	5.15
	20	0.15	0.29	0.56	0.85	1.24	1.67	2.16	2.72	3.33	3.99	4.70
	24	0.12	0.25	0.47	0.71	1.03	1.40	1.82	2.29	2.81	3.37	3.99
	28	0.11	0.21	0.40	0.61	0.89	1.20	1.57	1.97	2.42	2.92	3.45
	32	0.09	0.18	0.35	0.54	0.78	1.05	1.37	1.73	2.13	2.57	3.04
	36	0.08	0.16	0.31	0.48	0.69	0.94	1.22	1.54	1.90	2.29	2.72
	C′	2.94	5.89	11.3	17.1	25.1	33.8	44.4	55.9	69.2	83.5	100
6	2	1.63	2.71	3.75	4.77	5.77	6.77	7.76	8.75	9.74	10.7	11.7
	3	1.39	2.48	3.56	4.60	5.63	6.65	7.65	8.66	9.66	10.7	11.6
	4	1.18	2.23	3.32	4.39	5.45	6.48	7.51	8.52	9.53	10.5	11.5
	5	1.01	1.98	3.07	4.15	5.23	6.28	7.33	8.36	9.38	10.4	11.4
	6	0.88	1.75	2.81	3.90	4.98	6.06	7.12	8.17	9.21	10.2	11.3
	7	0.77	1.56	2.58	3.64	4.73	5.81	6.89	7.95	9.00	10.1	11.1
	8	0.69	1.40	2.36	3.40	4.47	5.56	6.64	7.72	8.78	9.84	10.9
	9	0.62	1.26	2.17	3.17	4.22	5.30	6.39	7.47	8.55	9.61	10.7
	10	0.56	1.15	2.01	2.96	3.98	5.05	6.13	7.22	8.30	9.38	10.4
	12	0.48	0.97	1.73	2.59	3.55	4.57	5.63	6.70	7.79	8.87	9.96
	14	0.41	0.83	1.51	2.28	3.17	4.13	5.15	6.20	7.28	8.36	9.44
	16	0.36	0.73	1.34	2.04	2.85	3.75	4.72	5.73	6.78	7.85	8.93
	18	0.32	0.65	1.21	1.83	2.59	3.42	4.34	5.31	6.32	7.36	8.42
	20	0.29	0.59	1.09	1.66	2.36	3.14	4.00	4.92	5.89	6.90	7.94
	24	0.24	0.49	0.92	1.40	2.00	2.68	3.44	4.27	5.15	6.09	7.06
	28	0.21	0.42	0.79	1.21	1.74	2.33	3.01	3.75	4.55	5.41	6.31
	32	0.18	0.37	0.70	1.06	1.53	2.06	2.67	3.33	4.06	4.85	5.68
	36	0.16	0.33	0.62	0.95	1.37	1.84	2.39	3.00	3.66	4.38	5.15
	C′	5.89	11.8	22.5	34.3	50.2	67.6	88.8	112	138	167	199

Table 7-7 (continued)
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 15°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

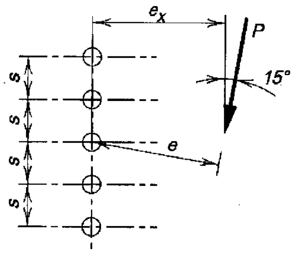
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n										
		2	3	4	5	6	7	8	9	10	11	12
3	2	1.15	2.20	3.28	4.34	5.39	6.42	7.45	8.46	9.47	10.5	11.5
	3	0.86	1.76	2.78	3.85	4.92	5.98	7.03	8.08	9.11	10.1	11.2
	4	0.67	1.42	2.35	3.36	4.41	5.48	6.55	7.61	8.67	9.72	10.8
	5	0.55	1.17	2.00	2.94	3.94	4.98	6.04	7.11	8.18	9.24	10.3
	6	0.47	0.99	1.73	2.58	3.52	4.52	5.55	6.61	7.67	8.74	9.81
	7	0.41	0.86	1.52	2.30	3.16	4.11	5.10	6.13	7.18	8.24	9.30
	8	0.36	0.75	1.35	2.06	2.86	3.74	4.69	5.68	6.70	7.74	8.80
	9	0.32	0.67	1.22	1.86	2.60	3.43	4.32	5.27	6.26	7.28	8.31
	10	0.29	0.61	1.10	1.69	2.38	3.16	4.00	4.90	5.85	6.84	7.85
	12	0.24	0.51	0.93	1.43	2.03	2.71	3.46	4.28	5.15	6.06	7.01
	14	0.21	0.43	0.81	1.24	1.76	2.37	3.04	3.78	4.57	5.41	6.30
	16	0.19	0.38	0.71	1.09	1.56	2.10	2.70	3.37	4.09	4.87	5.69
	18	0.17	0.34	0.63	0.97	1.39	1.88	2.43	3.04	3.70	4.42	5.18
	20	0.15	0.3	0.57	0.88	1.26	1.70	2.20	2.76	3.37	4.03	4.74
	24	0.12	0.25	0.48	0.73	1.06	1.43	1.86	2.33	2.86	3.43	4.04
	28	0.11	0.22	0.41	0.63	0.91	1.23	1.60	2.02	2.47	2.97	3.51
	32	0.09	0.19	0.36	0.55	0.80	1.08	1.41	1.77	2.18	2.62	3.10
	36	0.08	0.17	0.32	0.49	0.71	0.96	1.26	1.58	1.95	2.34	2.78
6	2	1.61	2.69	3.72	4.74	5.74	6.74	7.73	8.73	9.71	10.7	11.7
	3	1.36	2.45	3.52	4.56	5.59	6.60	7.61	8.61	9.61	10.6	11.6
	4	1.15	2.20	3.28	4.34	5.39	6.42	7.45	8.46	9.47	10.5	11.5
	5	0.98	1.96	3.03	4.10	5.16	6.21	7.25	8.28	9.30	10.3	11.3
	6	0.86	1.76	2.78	3.85	4.92	5.98	7.03	8.08	9.11	10.1	11.2
	7	0.75	1.57	2.55	3.60	4.66	5.73	6.80	7.85	8.90	9.94	11.0
	8	0.67	1.42	2.35	3.36	4.41	5.48	6.55	7.61	8.67	9.72	10.8
	9	0.61	1.29	2.16	3.14	4.17	5.23	6.30	7.36	8.43	9.49	10.5
	10	0.55	1.17	2.00	2.94	3.94	4.98	6.04	7.11	8.18	9.24	10.3
	12	0.47	0.99	1.73	2.58	3.52	4.52	5.55	6.61	7.67	8.74	9.81
	14	0.41	0.86	1.52	2.3	3.16	4.11	5.10	6.13	7.18	8.24	9.30
	16	0.36	0.75	1.35	2.06	2.86	3.74	4.69	5.68	6.70	7.74	8.80
	18	0.32	0.67	1.22	1.86	2.60	3.43	4.32	5.27	6.26	7.28	8.31
	20	0.29	0.61	1.10	1.69	2.38	3.16	4.00	4.90	5.85	6.84	7.85
	24	0.24	0.51	0.93	1.43	2.03	2.71	3.46	4.28	5.15	6.06	7.01
	28	0.21	0.43	0.81	1.24	1.76	2.37	3.04	3.78	4.57	5.41	6.30
	32	0.19	0.38	0.71	1.09	1.56	2.10	2.70	3.37	4.09	4.87	5.69
	36	0.17	0.34	0.63	0.97	1.39	1.88	2.43	3.04	3.70	4.42	5.18

Table 7-7 (continued)

Coefficients *C* for Eccentrically Loaded Bolt Groups

Angle = 30°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

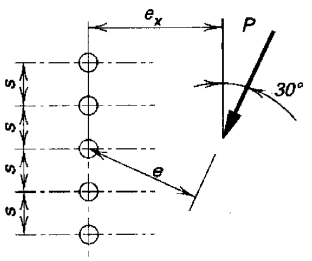
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
3	2	1.14	2.20	3.25	4.30	5.33	6.36	7.38	8.39	9.40	10.4	11.4
	3	0.86	1.80	2.79	3.83	4.87	5.92	6.96	7.99	9.02	10.0	11.1
	4	0.69	1.50	2.40	3.39	4.41	5.45	6.49	7.53	8.57	9.61	10.6
	5	0.57	1.27	2.08	3.00	3.98	4.99	6.02	7.06	8.11	9.15	10.2
	6	0.49	1.09	1.82	2.68	3.60	4.57	5.58	6.60	7.64	8.68	9.72
	7	0.43	0.95	1.61	2.40	3.27	4.20	5.17	6.17	7.18	8.21	9.25
	8	0.38	0.83	1.44	2.17	2.98	3.86	4.79	5.76	6.75	7.77	8.79
	9	0.34	0.75	1.30	1.98	2.74	3.57	4.46	5.39	6.35	7.34	8.35
	10	0.31	0.67	1.19	1.82	2.52	3.31	4.15	5.05	5.98	6.95	7.93
	12	0.26	0.56	1.01	1.55	2.17	2.87	3.64	4.46	5.33	6.24	7.17
	14	0.23	0.48	0.87	1.35	1.90	2.53	3.23	3.98	4.78	5.63	6.51
	16	0.20	0.42	0.77	1.20	1.69	2.26	2.89	3.58	4.33	5.11	5.94
	18	0.18	0.38	0.69	1.07	1.52	2.04	2.62	3.25	3.94	4.67	5.45
	20	0.16	0.34	0.62	0.97	1.37	1.85	2.38	2.97	3.61	4.30	5.02
	24	0.14	0.28	0.52	0.81	1.16	1.57	2.02	2.53	3.09	3.69	4.33
	28	0.12	0.24	0.45	0.70	1.00	1.36	1.75	2.20	2.69	3.22	3.79
	32	0.10	0.21	0.40	0.61	0.88	1.19	1.54	1.94	2.38	2.85	3.37
	36	0.09	0.19	0.35	0.55	0.78	1.07	1.38	1.74	2.13	2.56	3.03
6	2	1.59	2.66	3.69	4.70	5.71	6.70	7.70	8.69	9.68	10.7	11.7
	3	1.34	2.43	3.48	4.52	5.54	6.55	7.55	8.56	9.55	10.6	11.5
	4	1.14	2.20	3.25	4.30	5.33	6.36	7.38	8.39	9.40	10.4	11.4
	5	0.98	1.99	3.02	4.06	5.11	6.14	7.17	8.20	9.22	10.2	11.2
	6	0.86	1.80	2.79	3.83	4.87	5.92	6.96	7.99	9.02	10.0	11.1
	7	0.77	1.64	2.59	3.60	4.64	5.68	6.73	7.77	8.80	9.83	10.9
	8	0.69	1.50	2.40	3.39	4.41	5.45	6.49	7.53	8.57	9.61	10.6
	9	0.63	1.37	2.23	3.19	4.19	5.22	6.26	7.30	8.34	9.38	10.4
	10	0.57	1.27	2.08	3.00	3.98	4.99	6.02	7.06	8.11	9.15	10.2
	12	0.49	1.09	1.82	2.68	3.60	4.57	5.58	6.60	7.64	8.68	9.72
	14	0.43	0.95	1.61	2.40	3.27	4.20	5.17	6.17	7.18	8.21	9.25
	16	0.38	0.83	1.44	2.17	2.98	3.86	4.79	5.76	6.75	7.77	8.79
	18	0.34	0.75	1.30	1.98	2.74	3.57	4.46	5.39	6.35	7.34	8.35
	20	0.31	0.67	1.19	1.82	2.52	3.31	4.15	5.05	5.98	6.95	7.93
	24	0.26	0.56	1.01	1.55	2.17	2.87	3.64	4.46	5.33	6.24	7.17
	28	0.23	0.48	0.87	1.35	1.90	2.53	3.23	3.98	4.78	5.63	6.51
	32	0.20	0.42	0.77	1.20	1.69	2.26	2.89	3.58	4.33	5.11	5.94
	36	0.18	0.38	0.69	1.07	1.52	2.04	2.62	3.25	3.94	4.67	5.45

Table 7–7 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi r_n}$	$C_{min} = \frac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

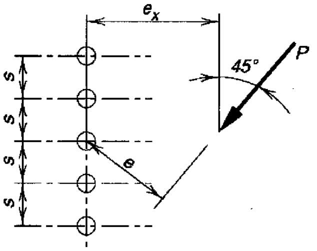
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
3	2	1.17	2.23	3.26	4.28	5.29	6.30	7.31	8.32	9.32	10.3	11.3
	3	0.92	1.89	2.87	3.87	4.88	5.90	6.91	7.93	8.94	9.95	11.0
	4	0.75	1.63	2.54	3.5	4.49	5.49	6.51	7.52	8.53	9.55	10.6
	5	0.64	1.42	2.25	3.17	4.13	5.11	6.11	7.11	8.12	9.14	10.2
	6	0.55	1.25	2.01	2.88	3.8	4.76	5.73	6.73	7.73	8.73	9.74
	7	0.49	1.11	1.81	2.63	3.51	4.43	5.38	6.36	7.34	8.34	9.34
	8	0.44	0.99	1.64	2.41	3.25	4.14	5.06	6.01	6.98	7.96	8.96
	9	0.40	0.90	1.49	2.22	3.02	3.87	4.77	5.69	6.64	7.61	8.58
	10	0.36	0.81	1.37	2.06	2.82	3.63	4.50	5.39	6.32	7.27	8.23
	12	0.31	0.68	1.17	1.79	2.47	3.22	4.02	4.87	5.74	6.65	7.58
	14	0.27	0.59	1.03	1.58	2.20	2.88	3.62	4.41	5.24	6.11	6.99
	16	0.24	0.52	0.91	1.41	1.97	2.60	3.29	4.03	4.81	5.63	6.48
	18	0.21	0.46	0.82	1.27	1.78	2.36	3.00	3.70	4.43	5.21	6.02
	20	0.19	0.41	0.74	1.16	1.62	2.16	2.76	3.41	4.1	4.84	5.61
	24	0.16	0.35	0.63	0.98	1.38	1.85	2.37	2.94	3.56	4.22	4.92
	28	0.14	0.30	0.54	0.85	1.19	1.61	2.08	2.58	3.14	3.73	4.37
	32	0.12	0.26	0.48	0.75	1.05	1.43	1.84	2.30	2.80	3.34	3.92
	36	0.11	0.23	0.43	0.67	0.94	1.28	1.65	2.07	2.53	3.02	3.55
6	2	1.57	2.64	3.66	4.67	5.67	6.66	7.66	8.65	9.64	10.6	11.6
	3	1.35	2.43	3.46	4.48	5.49	6.49	7.50	8.49	9.49	10.5	11.5
	4	1.17	2.23	3.26	4.28	5.29	6.30	7.31	8.32	9.32	10.3	11.3
	5	1.03	2.05	3.06	4.07	5.09	6.10	7.12	8.13	9.13	10.1	11.1
	6	0.92	1.89	2.87	3.87	4.88	5.90	6.91	7.93	8.94	9.95	11.0
	7	0.83	1.75	2.70	3.68	4.68	5.69	6.71	7.72	8.74	9.75	10.8
	8	0.75	1.63	2.54	3.50	4.49	5.49	6.51	7.52	8.53	9.55	10.6
	9	0.69	1.52	2.39	3.33	4.30	5.30	6.30	7.31	8.33	9.34	10.4
	10	0.64	1.42	2.25	3.17	4.13	5.11	6.11	7.11	8.12	9.14	10.2
	12	0.55	1.25	2.01	2.88	3.80	4.76	5.73	6.73	7.73	8.73	9.74
	14	0.49	1.11	1.81	2.63	3.51	4.43	5.38	6.36	7.34	8.34	9.34
	16	0.44	0.99	1.64	2.41	3.25	4.14	5.06	6.01	6.98	7.96	8.96
	18	0.40	0.9	1.49	2.22	3.02	3.87	4.77	5.69	6.64	7.61	8.58
	20	0.36	0.81	1.37	2.06	2.82	3.63	4.50	5.39	6.32	7.27	8.23
	24	0.31	0.68	1.17	1.79	2.47	3.22	4.02	4.87	5.74	6.65	7.58
	28	0.27	0.59	1.03	1.58	2.20	2.88	3.62	4.41	5.24	6.11	6.99
	32	0.24	0.52	0.91	1.41	1.97	2.60	3.29	4.03	4.81	5.63	6.48
	36	0.21	0.46	0.82	1.27	1.78	2.36	3.00	3.70	4.43	5.21	6.02

Table 7–7 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\quad \Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

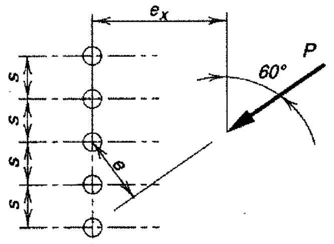
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
3	2	1.27	2.32	3.32	4.31	5.30	6.30	7.29	8.27	9.27	10.3	11.3
	3	1.05	2.05	3.02	4.00	4.98	5.97	6.96	7.94	8.94	9.93	10.9
	4	0.89	1.83	2.77	3.72	4.69	5.66	6.64	7.62	8.61	9.60	10.6
	5	0.77	1.65	2.54	3.47	4.41	5.37	6.34	7.32	8.29	9.28	10.3
	6	0.68	1.49	2.34	3.24	4.16	5.10	6.06	7.02	7.99	8.97	9.95
	7	0.61	1.37	2.17	3.03	3.93	4.85	5.79	6.74	7.71	8.67	9.64
	8	0.56	1.26	2.01	2.83	3.71	4.61	5.54	6.48	7.43	8.39	9.35
	9	0.51	1.16	1.87	2.66	3.51	4.39	5.30	6.23	7.17	8.12	9.07
	10	0.47	1.07	1.74	2.50	3.32	4.19	5.08	5.99	6.92	7.86	8.81
	12	0.40	0.93	1.52	2.22	3.00	3.82	4.67	5.55	6.45	7.37	8.30
	14	0.35	0.81	1.35	2.00	2.73	3.50	4.32	5.16	6.03	6.92	7.83
	16	0.32	0.72	1.21	1.81	2.49	3.23	4.00	4.81	5.65	6.51	7.40
	18	0.29	0.65	1.09	1.66	2.30	2.98	3.72	4.50	5.31	6.14	7.00
	20	0.26	0.58	1.00	1.53	2.12	2.77	3.47	4.21	4.99	5.80	6.63
	24	0.22	0.49	0.85	1.32	1.84	2.41	3.05	3.73	4.45	5.21	5.99
	28	0.19	0.42	0.74	1.15	1.61	2.13	2.71	3.34	4.00	4.70	5.44
	32	0.17	0.37	0.65	1.02	1.43	1.91	2.44	3.02	3.63	4.28	4.97
	36	0.15	0.33	0.59	0.92	1.29	1.72	2.21	2.74	3.31	3.92	4.57
6	2	1.60	2.65	3.65	4.64	5.64	6.63	7.62	8.61	9.60	10.6	11.6
	3	1.42	2.48	3.48	4.48	5.47	6.46	7.45	8.44	9.44	10.4	11.4
	4	1.27	2.32	3.32	4.31	5.30	6.30	7.29	8.27	9.27	10.3	11.3
	5	1.15	2.18	3.17	4.15	5.14	6.13	7.12	8.11	9.10	10.1	11.1
	6	1.05	2.05	3.02	4.00	4.98	5.97	6.96	7.94	8.94	9.93	10.9
	7	0.96	1.93	2.89	3.86	4.83	5.81	6.80	7.78	8.77	9.76	10.8
	8	0.89	1.83	2.77	3.72	4.69	5.66	6.64	7.62	8.61	9.60	10.6
	9	0.83	1.73	2.65	3.59	4.55	5.51	6.49	7.47	8.45	9.43	10.4
	10	0.77	1.65	2.54	3.47	4.41	5.37	6.34	7.32	8.29	9.28	10.3
	12	0.68	1.49	2.34	3.24	4.16	5.10	6.06	7.02	7.99	8.97	9.95
	14	0.61	1.37	2.17	3.03	3.93	4.85	5.79	6.74	7.71	8.67	9.64
	16	0.56	1.26	2.01	2.83	3.71	4.61	5.54	6.48	7.43	8.39	9.35
	18	0.51	1.16	1.87	2.66	3.51	4.39	5.30	6.23	7.17	8.12	9.07
	20	0.47	1.07	1.74	2.50	3.32	4.19	5.08	5.99	6.92	7.86	8.81
	24	0.40	0.93	1.52	2.22	3.00	3.82	4.67	5.55	6.45	7.37	8.30
	28	0.35	0.81	1.35	2.00	2.73	3.50	4.32	5.16	6.03	6.92	7.83
	32	0.32	0.72	1.21	1.81	2.49	3.23	4.00	4.81	5.65	6.51	7.40
	36	0.29	0.65	1.09	1.66	2.30	2.98	3.72	4.50	5.31	6.14	7.00

Table 7-7 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

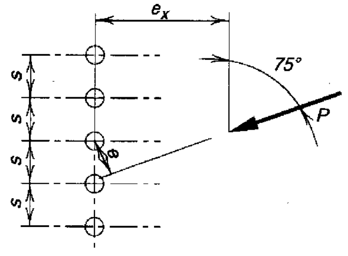
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	*e*$_x$, in.	Number of Bolts in One Vertical Row, *n*										
		2	3	4	5	6	7	8	9	10	11	12
3	2	1.49	2.51	3.49	4.46	5.44	6.42	7.40	8.38	9.36	10.3	11.3
	3	1.32	2.33	3.30	4.27	5.24	6.21	7.18	8.15	9.13	10.1	11.1
	4	1.18	2.18	3.14	4.09	5.05	6.01	6.98	7.95	8.92	9.89	10.9
	5	1.07	2.04	2.99	3.93	4.88	5.84	6.79	7.75	8.72	9.68	10.7
	6	0.98	1.92	2.85	3.79	4.73	5.67	6.62	7.57	8.53	9.49	10.5
	7	0.90	1.82	2.73	3.65	4.58	5.52	6.46	7.40	8.36	9.31	10.3
	8	0.84	1.72	2.62	3.52	4.44	5.37	6.30	7.24	8.19	9.14	10.1
	9	0.78	1.63	2.51	3.40	4.31	5.23	6.16	7.09	8.03	8.97	9.92
	10	0.73	1.55	2.41	3.29	4.19	5.10	6.02	6.94	7.88	8.81	9.76
	12	0.65	1.41	2.23	3.08	3.95	4.84	5.75	6.66	7.59	8.51	9.45
	14	0.58	1.30	2.06	2.88	3.73	4.60	5.50	6.40	7.31	8.23	9.16
	16	0.53	1.20	1.92	2.70	3.52	4.38	5.26	6.15	7.05	7.96	8.88
	18	0.48	1.11	1.78	2.53	3.33	4.17	5.03	5.91	6.80	7.70	8.61
	20	0.44	1.03	1.66	2.38	3.16	3.97	4.82	5.69	6.56	7.45	8.35
	24	0.38	0.89	1.46	2.12	2.85	3.63	4.44	5.27	6.13	6.99	7.87
	28	0.34	0.79	1.29	1.90	2.59	3.33	4.11	4.91	5.73	6.57	7.43
	32	0.30	0.70	1.16	1.73	2.38	3.08	3.81	4.58	5.37	6.19	7.02
	36	0.27	0.62	1.05	1.58	2.19	2.85	3.55	4.28	5.05	5.84	6.65
6	2	1.71	2.72	3.70	4.69	5.67	6.66	7.64	8.79	9.78	10.8	11.7
	3	1.60	2.61	3.59	4.57	5.55	6.53	7.52	8.50	9.48	10.5	11.5
	4	1.49	2.51	3.49	4.46	5.44	6.42	7.40	8.38	9.36	10.3	11.3
	5	1.40	2.42	3.39	4.37	5.34	6.31	7.29	8.26	9.24	10.2	11.2
	6	1.32	2.33	3.30	4.27	5.24	6.21	7.18	8.15	9.13	10.1	11.1
	7	1.25	2.25	3.22	4.18	5.14	6.11	7.07	8.05	9.01	10.0	11.0
	8	1.18	2.18	3.14	4.09	5.05	6.01	6.98	7.95	8.92	9.89	10.9
	9	1.13	2.11	3.06	4.01	4.97	5.92	6.88	7.85	8.81	9.78	10.8
	10	1.07	2.04	2.99	3.93	4.88	5.84	6.79	7.75	8.72	9.68	10.7
	12	0.98	1.92	2.85	3.79	4.73	5.67	6.62	7.57	8.53	9.49	10.5
	14	0.90	1.82	2.73	3.65	4.58	5.52	6.46	7.40	8.36	9.31	10.3
	16	0.84	1.72	2.62	3.52	4.44	5.37	6.30	7.24	8.19	9.14	10.1
	18	0.78	1.63	2.51	3.40	4.31	5.23	6.16	7.09	8.03	8.97	9.92
	20	0.73	1.55	2.41	3.29	4.19	5.10	6.02	6.94	7.88	8.81	9.76
	24	0.65	1.41	2.23	3.08	3.95	4.84	5.75	6.66	7.59	8.51	9.45
	28	0.58	1.30	2.06	2.88	3.73	4.60	5.50	6.40	7.31	8.23	9.16
	32	0.53	1.20	1.92	2.70	3.52	4.38	5.26	6.15	7.05	7.96	8.88
	36	0.48	1.11	1.78	2.53	3.33	4.17	5.03	5.91	6.80	7.70	8.61

Table 7-8
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

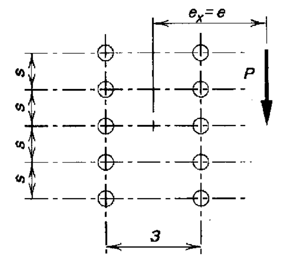
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.84	2.54	4.48	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.0
	3	0.65	2.03	3.68	5.67	7.77	9.91	12.1	14.2	16.3	18.3	20.4	22.5
	4	0.54	1.67	3.06	4.86	6.84	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	5	0.45	1.42	2.59	4.21	6.01	8.00	10.1	12.2	14.4	16.5	18.7	20.8
	6	0.39	1.22	2.25	3.69	5.32	7.17	9.16	11.2	13.4	15.5	17.7	19.8
	7	0.35	1.08	1.99	3.27	4.74	6.46	8.33	10.3	12.4	14.5	16.7	18.8
	8	0.31	0.96	1.78	2.93	4.27	5.86	7.60	9.50	11.5	13.6	15.7	17.8
	9	0.28	0.86	1.60	2.65	3.87	5.34	6.97	8.75	10.7	12.7	14.7	16.8
	10	0.26	0.78	1.46	2.42	3.53	4.90	6.42	8.10	9.91	11.8	13.8	15.9
	12	0.22	0.66	1.24	2.06	3.01	4.19	5.51	7.01	8.63	10.4	12.2	14.2
	14	0.19	0.57	1.08	1.78	2.62	3.66	4.82	6.15	7.61	9.19	10.9	12.7
	16	0.17	0.51	0.95	1.57	2.32	3.24	4.27	5.47	6.79	8.23	9.78	11.4
	18	0.15	0.45	0.85	1.41	2.07	2.90	3.83	4.92	6.11	7.43	8.85	10.4
	20	0.14	0.41	0.77	1.27	1.88	2.63	3.48	4.47	5.55	6.76	8.07	9.48
	24	0.12	0.34	0.65	1.07	1.58	2.21	2.93	3.77	4.69	5.72	6.85	8.06
	28	0.10	0.29	0.56	0.92	1.36	1.90	2.53	3.25	4.05	4.95	5.93	7.00
	32	0.09	0.26	0.49	0.80	1.19	1.67	2.22	2.86	3.57	4.36	5.23	6.18
	36	0.08	0.23	0.43	0.72	1.06	1.49	1.98	2.55	3.18	3.90	4.67	5.52
	C′	2.94	8.33	15.8	26.0	38.7	54.2	72.2	93.1	117	143	172	204
6	2	0.84	3.24	5.39	7.47	9.51	11.5	13.5	15.5	17.5	19.5	21.5	23.4
	3	0.65	2.79	4.93	7.08	9.17	11.2	13.3	15.3	17.3	19.3	21.3	23.3
	4	0.54	2.41	4.44	6.60	8.75	10.9	12.9	15.0	17.0	19.1	21.1	23.1
	5	0.45	2.10	3.97	6.11	8.27	10.4	12.5	14.6	16.7	18.7	20.8	22.8
	6	0.39	1.85	3.55	5.62	7.77	9.93	12.1	14.2	16.3	18.4	20.4	22.5
	7	0.35	1.64	3.18	5.17	7.27	9.43	11.6	13.7	15.9	18.0	20.1	22.1
	8	0.31	1.47	2.87	4.75	6.79	8.92	11.1	13.3	15.4	17.5	19.6	21.7
	9	0.28	1.34	2.61	4.39	6.34	8.43	10.6	12.7	14.9	17.1	19.2	21.3
	10	0.26	1.22	2.39	4.06	5.92	7.96	10.1	12.2	14.4	16.6	18.7	20.9
	12	0.22	1.04	2.04	3.52	5.20	7.10	9.12	11.2	13.4	15.5	17.7	19.9
	14	0.19	0.90	1.77	3.09	4.61	6.36	8.27	10.3	12.4	14.5	16.7	18.9
	16	0.17	0.80	1.57	2.75	4.12	5.74	7.52	9.44	11.5	13.5	15.7	17.8
	18	0.15	0.71	1.41	2.48	3.72	5.21	6.87	8.68	10.6	12.6	14.7	16.8
	20	0.14	0.64	1.28	2.25	3.38	4.77	6.31	8.02	9.85	11.8	13.8	15.9
	24	0.12	0.54	1.07	1.90	2.86	4.06	5.40	6.91	8.55	10.3	12.2	14.1
	28	0.10	0.46	0.93	1.64	2.47	3.52	4.70	6.05	7.52	9.12	10.8	12.6
	32	0.09	0.41	0.81	1.44	2.18	3.11	4.16	5.37	6.69	8.15	9.71	11.4
	36	0.08	0.36	0.73	1.29	1.94	2.78	3.72	4.81	6.02	7.34	8.78	10.3
	C′	2.94	13.2	26.5	47.0	71.4	103	138	180	226	279	337	400

Table 7-8 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

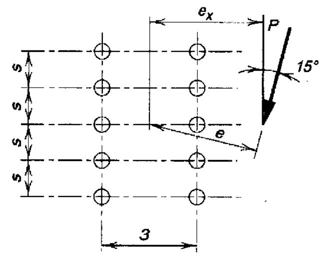
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.87	2.54	4.47	6.54	8.63	10.7	12.8	14.8	16.9	18.9	20.9	22.9
	3	0.68	2.04	3.71	5.63	7.69	9.80	11.9	14.0	16.1	18.2	20.2	22.3
	4	0.55	1.69	3.11	4.85	6.79	8.84	10.9	13.1	15.2	17.3	19.4	21.5
	5	0.47	1.44	2.66	4.21	6.00	7.94	9.98	12.1	14.2	16.3	18.4	20.5
	6	0.41	1.25	2.31	3.70	5.34	7.15	9.09	11.1	13.2	15.3	17.4	19.6
	7	0.36	1.10	2.04	3.29	4.79	6.46	8.30	10.2	12.3	14.3	16.4	18.6
	8	0.32	0.98	1.83	2.96	4.32	5.87	7.60	9.45	11.4	13.4	15.5	17.6
	9	0.29	0.88	1.65	2.68	3.94	5.37	6.99	8.74	10.6	12.6	14.6	16.6
	10	0.27	0.81	1.51	2.45	3.61	4.93	6.45	8.11	9.88	11.8	13.7	15.7
	12	0.23	0.68	1.28	2.09	3.08	4.24	5.58	7.05	8.66	10.4	12.2	14.1
	14	0.20	0.59	1.11	1.82	2.69	3.71	4.90	6.21	7.67	9.23	10.9	12.7
	16	0.17	0.52	0.98	1.61	2.38	3.29	4.36	5.54	6.86	8.29	9.83	11.5
	18	0.16	0.47	0.88	1.44	2.13	2.96	3.92	4.99	6.20	7.51	8.93	10.4
	20	0.14	0.42	0.79	1.31	1.93	2.68	3.56	4.54	5.65	6.85	8.17	9.57
	24	0.12	0.35	0.67	1.10	1.62	2.26	3.00	3.84	4.79	5.82	6.96	8.17
	28	0.10	0.30	0.57	0.94	1.40	1.95	2.60	3.32	4.15	5.05	6.05	7.12
	32	0.09	0.27	0.50	0.83	1.23	1.72	2.28	2.93	3.66	4.46	5.34	6.29
	36	0.08	0.24	0.45	0.74	1.10	1.53	2.04	2.61	3.27	3.98	4.78	5.64
6	2	0.87	3.21	5.35	7.42	9.45	11.5	13.5	15.5	17.4	19.4	21.4	23.4
	3	0.68	2.76	4.88	7.00	9.09	11.1	13.2	15.2	17.2	19.2	21.2	23.2
	4	0.55	2.38	4.40	6.53	8.65	10.7	12.8	14.9	16.9	18.9	20.9	22.9
	5	0.47	2.07	3.96	6.04	8.17	10.3	12.4	14.5	16.5	18.6	20.6	22.6
	6	0.41	1.83	3.56	5.56	7.67	9.80	11.9	14.0	16.1	18.2	20.3	22.3
	7	0.36	1.63	3.22	5.12	7.19	9.30	11.4	13.6	15.7	17.8	19.9	21.9
	8	0.32	1.47	2.92	4.73	6.72	8.81	10.9	13.1	15.2	17.3	19.4	21.5
	9	0.29	1.34	2.66	4.37	6.29	8.33	10.4	12.6	14.7	16.8	18.9	21.0
	10	0.27	1.23	2.45	4.05	5.90	7.88	9.95	12.1	14.2	16.3	18.5	20.6
	12	0.23	1.05	2.09	3.53	5.21	7.06	9.04	11.1	13.2	15.3	17.5	19.6
	14	0.2	0.91	1.83	3.11	4.64	6.35	8.22	10.2	12.2	14.3	16.5	18.6
	16	0.17	0.81	1.62	2.78	4.17	5.75	7.51	9.38	11.4	13.4	15.5	17.6
	18	0.16	0.72	1.45	2.50	3.77	5.24	6.88	8.66	10.5	12.5	14.5	16.6
	20	0.14	0.66	1.32	2.28	3.45	4.80	6.34	8.02	9.82	11.7	13.7	15.7
	24	0.12	0.55	1.11	1.93	2.93	4.10	5.46	6.95	8.57	10.3	12.1	14.0
	28	0.10	0.48	0.96	1.67	2.54	3.57	4.78	6.11	7.58	9.15	10.8	12.6
	32	0.09	0.42	0.84	1.47	2.24	3.16	4.24	5.44	6.77	8.21	9.75	11.4
	36	0.08	0.37	0.75	1.32	2.00	2.83	3.80	4.89	6.10	7.42	8.85	10.4

Table 7-8 (continued)

Coefficients *C* for Eccentrically Loaded Bolt Groups

Angle = 30°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

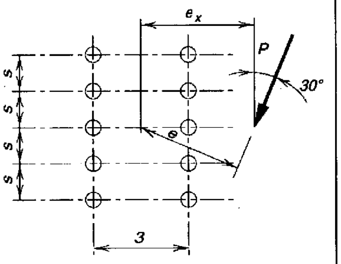
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	0.97	2.60	4.52	6.54	8.59	10.6	12.7	14.7	16.7	18.8	20.8	22.8
	3	0.75	2.12	3.83	5.71	7.71	9.75	11.8	13.9	15.9	18.0	20.0	22.1
	4	0.62	1.78	3.29	4.99	6.88	8.87	10.9	13.0	15.1	17.1	19.2	21.3
	5	0.52	1.53	2.85	4.39	6.16	8.06	10.0	12.1	14.1	16.2	18.3	20.4
	6	0.45	1.34	2.51	3.89	5.54	7.33	9.23	11.2	13.2	15.3	17.3	19.4
	7	0.40	1.19	2.23	3.48	5.01	6.70	8.51	10.4	12.4	14.4	16.4	18.5
	8	0.36	1.07	2.00	3.15	4.57	6.14	7.86	9.68	11.6	13.6	15.6	17.6
	9	0.32	0.97	1.81	2.87	4.19	5.66	7.28	9.02	10.9	12.8	14.7	16.7
	10	0.30	0.88	1.66	2.64	3.87	5.24	6.77	8.43	10.2	12.0	13.9	15.9
	12	0.25	0.75	1.41	2.27	3.34	4.54	5.92	7.43	9.04	10.8	12.5	14.4
	14	0.22	0.65	1.23	1.98	2.93	3.99	5.24	6.61	8.09	9.67	11.4	13.1
	16	0.19	0.58	1.08	1.76	2.60	3.56	4.69	5.94	7.30	8.77	10.3	12.0
	18	0.17	0.52	0.97	1.58	2.34	3.21	4.24	5.38	6.64	8.0	9.45	11.0
	20	0.16	0.47	0.88	1.43	2.12	2.92	3.87	4.92	6.08	7.3	8.70	10.1
	24	0.13	0.39	0.74	1.21	1.79	2.48	3.29	4.18	5.19	6.3	7.48	8.75
	28	0.12	0.34	0.64	1.04	1.55	2.14	2.85	3.63	4.52	5.5	6.54	7.68
	32	0.10	0.30	0.56	0.92	1.36	1.89	2.51	3.21	4.00	4.9	5.81	6.83
	36	0.09	0.26	0.50	0.82	1.21	1.69	2.25	2.87	3.59	4.4	5.22	6.15
6	2	0.97	3.20	5.31	7.37	9.39	11.4	13.4	15.4	17.4	19.4	21.3	23.3
	3	0.75	2.75	4.86	6.95	9.01	11.1	13.1	15.1	17.1	19.1	21.1	23.1
	4	0.62	2.39	4.42	6.49	8.57	10.6	12.7	14.7	16.8	18.8	20.8	22.8
	5	0.52	2.10	4.02	6.04	8.11	10.2	12.3	14.3	16.4	18.4	20.4	22.5
	6	0.45	1.87	3.67	5.61	7.66	9.73	11.8	13.9	16.0	18.0	20.1	22.1
	7	0.40	1.69	3.36	5.21	7.21	9.27	11.4	13.4	15.5	17.6	19.6	21.7
	8	0.36	1.53	3.08	4.84	6.79	8.82	10.9	13.0	15.1	17.1	19.2	21.3
	9	0.32	1.40	2.84	4.51	6.40	8.39	10.4	12.5	14.6	16.7	18.7	20.8
	10	0.30	1.29	2.63	4.21	6.04	7.98	9.99	12.0	14.1	16.2	18.3	20.4
	12	0.25	1.12	2.28	3.70	5.39	7.23	9.16	11.2	13.2	15.3	17.3	19.4
	14	0.22	0.98	2.00	3.29	4.86	6.57	8.41	10.3	12.3	14.4	16.4	18.5
	16	0.19	0.87	1.78	2.95	4.40	6.01	7.75	9.6	11.5	13.5	15.5	17.6
	18	0.17	0.79	1.60	2.68	4.02	5.52	7.17	8.9	10.8	12.7	14.7	16.7
	20	0.16	0.71	1.45	2.45	3.70	5.09	6.65	8.3	10.1	12.0	13.9	15.9
	24	0.13	0.60	1.23	2.08	3.17	4.39	5.79	7.3	8.95	10.7	12.5	14.4
	28	0.12	0.52	1.06	1.82	2.77	3.85	5.11	6.5	7.99	9.59	11.3	13.0
	32	0.10	0.46	0.93	1.61	2.45	3.42	4.56	5.8	7.20	8.68	10.3	11.9
	36	0.09	0.41	0.83	1.44	2.20	3.08	4.12	5.3	6.53	7.91	9.37	10.9

Table 7-8 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

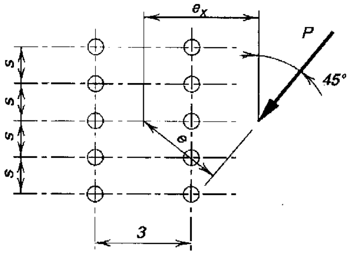
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.17	2.79	4.67	6.62	8.61	10.6	12.6	14.6	16.6	18.6	20.6	22.6
	3	0.92	2.32	4.06	5.92	7.86	9.83	11.8	13.9	15.9	17.9	19.9	21.9
	4	0.75	1.99	3.57	5.31	7.16	9.09	11.1	13.1	15.1	17.1	19.1	21.1
	5	0.64	1.74	3.17	4.78	6.53	8.39	10.3	12.3	14.3	16.3	18.3	20.3
	6	0.55	1.54	2.84	4.33	5.98	7.76	9.63	11.6	13.5	15.5	17.5	19.5
	7	0.49	1.38	2.57	3.93	5.49	7.20	9.00	10.9	12.8	14.8	16.7	18.7
	8	0.44	1.25	2.33	3.60	5.06	6.70	8.43	10.3	12.1	14.0	16.0	18.0
	9	0.40	1.14	2.13	3.31	4.69	6.25	7.91	9.67	11.5	13.4	15.3	17.2
	10	0.36	1.05	1.96	3.06	4.36	5.85	7.44	9.14	10.9	12.7	14.6	16.5
	12	0.31	0.90	1.68	2.65	3.83	5.17	6.63	8.20	9.86	11.6	13.4	15.2
	14	0.27	0.78	1.47	2.33	3.40	4.61	5.95	7.41	8.97	10.6	12.3	14.1
	16	0.24	0.69	1.31	2.08	3.05	4.16	5.38	6.74	8.20	9.75	11.4	13.1
	18	0.21	0.62	1.17	1.88	2.76	3.77	4.91	6.18	7.55	9.00	10.5	12.1
	20	0.19	0.56	1.06	1.71	2.52	3.45	4.51	5.69	6.97	8.34	9.80	11.3
	24	0.16	0.48	0.90	1.45	2.14	2.94	3.87	4.91	6.04	7.26	8.57	9.95
	28	0.14	0.41	0.77	1.26	1.86	2.56	3.38	4.30	5.30	6.41	7.59	8.85
	32	0.12	0.36	0.68	1.11	1.64	2.27	3.00	3.82	4.73	5.73	6.80	7.94
	36	0.11	0.32	0.61	0.99	1.47	2.03	2.70	3.44	4.26	5.17	6.15	7.20
6	2	1.17	3.24	5.30	7.32	9.33	11.3	13.3	15.3	17.3	19.3	21.3	23.2
	3	0.92	2.84	4.90	6.93	8.96	11.0	13.0	15.0	17.0	19.0	21.0	23.0
	4	0.75	2.51	4.52	6.53	8.56	10.6	12.6	14.6	16.6	18.6	20.6	22.6
	5	0.64	2.24	4.17	6.15	8.15	10.2	12.2	14.2	16.2	18.3	20.3	22.3
	6	0.55	2.03	3.86	5.78	7.76	9.77	11.8	13.8	15.8	17.9	19.9	21.9
	7	0.49	1.85	3.59	5.45	7.39	9.38	11.4	13.4	15.4	17.5	19.5	21.5
	8	0.44	1.70	3.35	5.13	7.03	9.00	11.0	13.0	15.0	17.1	19.1	21.1
	9	0.40	1.57	3.13	4.85	6.70	8.63	10.6	12.6	14.6	16.7	18.7	20.7
	10	0.36	1.46	2.94	4.58	6.38	8.28	10.2	12.2	14.2	16.3	18.3	20.3
	12	0.31	1.28	2.60	4.11	5.81	7.64	9.54	11.5	13.5	15.5	17.5	19.5
	14	0.27	1.13	2.32	3.71	5.31	7.06	8.89	10.8	12.7	14.7	16.7	18.7
	16	0.24	1.01	2.09	3.36	4.88	6.55	8.31	10.2	12.0	14.0	15.9	17.9
	18	0.21	0.92	1.90	3.07	4.50	6.09	7.78	9.56	11.4	13.3	15.2	17.2
	20	0.19	0.84	1.73	2.83	4.18	5.69	7.31	9.02	10.8	12.7	14.6	16.5
	24	0.16	0.72	1.47	2.43	3.64	5.00	6.48	8.08	9.76	11.5	13.3	15.2
	28	0.14	0.62	1.28	2.13	3.22	4.45	5.80	7.28	8.86	10.5	12.2	14.0
	32	0.12	0.55	1.13	1.90	2.88	3.99	5.24	6.62	8.09	9.65	11.3	13.0
	36	0.11	0.49	1.01	1.71	2.61	3.62	4.77	6.05	7.43	8.90	10.4	12.1

Table 7-8 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

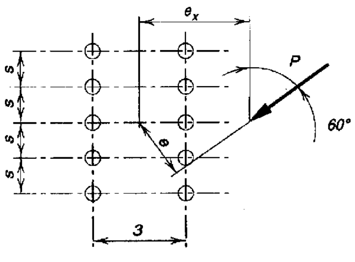
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.51	3.17	4.97	6.85	8.77	10.7	12.7	14.6	16.6	18.6	20.6	22.5
	3	1.24	2.76	4.47	6.30	8.19	10.1	12.0	14.0	16.0	17.9	19.9	21.9
	4	1.04	2.43	4.04	5.81	7.65	9.53	11.5	13.4	15.3	17.3	19.3	21.2
	5	0.89	2.16	3.70	5.39	7.17	9.01	10.9	12.8	14.7	16.7	18.6	20.6
	6	0.77	1.95	3.40	5.01	6.73	8.52	10.4	12.3	14.2	16.1	18.0	20.0
	7	0.68	1.77	3.13	4.67	6.33	8.07	9.88	11.7	13.6	15.5	17.4	19.4
	8	0.61	1.62	2.90	4.37	5.96	7.65	9.42	11.2	13.1	15.0	16.9	18.8
	9	0.56	1.49	2.70	4.09	5.62	7.26	8.98	10.8	12.6	14.5	16.3	18.2
	10	0.51	1.38	2.52	3.84	5.31	6.89	8.58	10.3	12.1	14.0	15.8	17.7
	12	0.43	1.20	2.21	3.40	4.76	6.25	7.85	9.53	11.3	13.0	14.9	16.7
	14	0.38	1.06	1.96	3.05	4.30	5.71	7.23	8.83	10.5	12.2	14.0	15.8
	16	0.34	0.95	1.76	2.75	3.92	5.24	6.68	8.20	9.79	11.5	13.2	14.9
	18	0.30	0.85	1.60	2.51	3.59	4.84	6.19	7.64	9.16	10.8	12.4	14.1
	20	0.27	0.78	1.46	2.30	3.32	4.48	5.76	7.14	8.60	10.1	11.7	13.4
	24	0.23	0.66	1.24	1.97	2.87	3.90	5.04	6.29	7.64	9.06	10.6	12.1
	28	0.20	0.57	1.07	1.72	2.52	3.44	4.47	5.61	6.85	8.17	9.55	11.0
	32	0.18	0.50	0.95	1.52	2.24	3.07	4.01	5.06	6.20	7.41	8.70	10.1
	36	0.16	0.45	0.85	1.37	2.02	2.77	3.63	4.59	5.65	6.77	7.98	9.26
6	2	1.51	3.39	5.36	7.33	9.31	11.3	13.3	15.2	17.2	19.2	21.2	23.2
	3	1.24	3.08	5.04	7.01	8.98	11.0	12.9	14.9	16.9	18.9	20.9	22.8
	4	1.04	2.80	4.73	6.69	8.66	10.6	12.6	14.6	16.6	18.6	20.5	22.5
	5	0.89	2.57	4.45	6.39	8.35	10.3	12.3	14.3	16.2	18.2	20.2	22.2
	6	0.77	2.37	4.20	6.11	8.05	10.0	12.0	13.9	15.9	17.9	19.9	21.8
	7	0.68	2.19	3.98	5.85	7.76	9.70	11.7	13.6	15.6	17.6	19.5	21.5
	8	0.61	2.04	3.77	5.61	7.49	9.41	11.4	13.3	15.3	17.2	19.2	21.2
	9	0.56	1.91	3.59	5.38	7.24	9.13	11.1	13.0	15.0	16.9	18.9	20.9
	10	0.51	1.80	3.42	5.17	7.00	8.87	10.8	12.7	14.7	16.6	18.6	20.5
	12	0.43	1.60	3.11	4.78	6.54	8.37	10.2	12.1	14.1	16.0	18.0	19.9
	14	0.38	1.44	2.85	4.43	6.13	7.91	9.74	11.6	13.5	15.4	17.4	19.3
	16	0.34	1.31	2.63	4.12	5.74	7.48	9.27	11.1	13.0	14.9	16.8	18.7
	18	0.30	1.20	2.43	3.84	5.40	7.08	8.84	10.7	12.5	14.4	16.3	18.2
	20	0.27	1.10	2.26	3.58	5.08	6.71	8.43	10.2	12.0	13.9	15.7	17.6
	24	0.23	0.95	1.97	3.15	4.53	6.06	7.69	9.39	11.2	12.9	14.8	16.6
	28	0.20	0.84	1.73	2.80	4.08	5.52	7.06	8.68	10.4	12.1	13.9	15.7
	32	0.18	0.74	1.54	2.52	3.71	5.05	6.51	8.05	9.66	11.3	13.1	14.8
	36	0.16	0.67	1.39	2.28	3.39	4.65	6.02	7.49	9.03	10.7	12.3	14.0

Table 7-8 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75 \qquad \Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

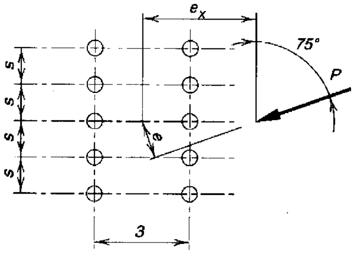
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.84	3.63	5.44	7.29	9.17	11.1	13.0	14.9	16.9	18.8	20.8	22.7
	3	1.71	3.41	5.17	6.97	8.82	10.7	12.6	14.5	16.4	18.4	20.3	22.3
	4	1.57	3.19	4.90	6.67	8.50	10.4	12.2	14.1	16.0	18.0	19.9	21.8
	5	1.44	2.98	4.65	6.39	8.19	10.0	11.9	13.8	15.7	17.6	19.5	21.4
	6	1.31	2.79	4.41	6.12	7.90	9.71	11.6	13.4	15.3	17.2	19.1	21.0
	7	1.20	2.61	4.19	5.88	7.62	9.42	11.3	13.1	15.0	16.9	18.8	20.7
	8	1.10	2.45	3.99	5.65	7.37	9.14	11.0	12.8	14.7	16.5	18.4	20.3
	9	1.01	2.31	3.81	5.43	7.14	8.89	10.7	12.5	14.3	16.2	18.1	20.0
	10	0.93	2.18	3.63	5.23	6.91	8.65	10.4	12.2	14.1	15.9	17.8	19.6
	12	0.81	1.95	3.33	4.86	6.49	8.19	9.94	11.7	13.5	15.3	17.2	19.0
	14	0.71	1.77	3.06	4.53	6.11	7.76	9.47	11.2	13.0	14.8	16.6	18.4
	16	0.63	1.61	2.83	4.23	5.75	7.36	9.03	10.8	12.5	14.3	16.1	17.9
	18	0.57	1.48	2.63	3.96	5.42	6.98	8.61	10.3	12.0	13.8	15.6	17.4
	20	0.52	1.36	2.45	3.72	5.12	6.63	8.23	9.88	11.6	13.3	15.1	16.9
	24	0.44	1.18	2.15	3.30	4.60	6.02	7.53	9.12	10.8	12.4	14.2	15.9
	28	0.38	1.04	1.91	2.95	4.16	5.49	6.93	8.45	10.0	11.7	13.3	15.0
	32	0.34	0.92	1.71	2.67	3.78	5.04	6.41	7.86	9.37	10.9	12.6	14.2
	36	0.30	0.83	1.55	2.43	3.47	4.65	5.94	7.32	8.78	10.3	11.9	13.5
6	2	1.84	3.66	5.55	7.48	9.42	11.4	13.3	15.3	17.6	19.6	21.5	23.5
	3	1.71	3.49	5.36	7.27	9.20	11.2	13.1	15.1	17.0	19.0	21.0	22.9
	4	1.57	3.32	5.18	7.08	9.00	10.9	12.9	14.8	16.8	18.7	20.7	22.7
	5	1.44	3.16	5.01	6.89	8.81	10.7	12.7	14.6	16.6	18.5	20.5	22.4
	6	1.31	3.02	4.84	6.72	8.62	10.5	12.5	14.4	16.3	18.3	20.2	22.2
	7	1.20	2.88	4.69	6.55	8.44	10.4	12.3	14.2	16.1	18.1	20.0	22.0
	8	1.10	2.75	4.54	6.39	8.27	10.2	12.1	14.0	15.9	17.9	19.8	21.8
	9	1.01	2.63	4.40	6.24	8.11	10.0	11.9	13.8	15.7	17.7	19.6	21.5
	10	0.93	2.52	4.27	6.09	7.95	9.83	11.7	13.6	15.6	17.5	19.4	21.3
	12	0.81	2.32	4.03	5.82	7.66	9.52	11.4	13.3	15.2	17.1	19.0	20.9
	14	0.71	2.15	3.82	5.57	7.38	9.22	11.1	13.0	14.9	16.7	18.7	20.6
	16	0.63	2.00	3.62	5.35	7.13	8.95	10.8	12.7	14.5	16.4	18.3	20.2
	18	0.57	1.87	3.44	5.14	6.90	8.69	10.5	12.4	14.2	16.1	18.0	19.9
	20	0.52	1.75	3.28	4.94	6.67	8.45	10.3	12.1	13.9	15.8	17.7	19.5
	24	0.44	1.55	2.98	4.57	6.24	7.98	9.75	11.6	13.4	15.2	17.1	18.9
	28	0.38	1.40	2.74	4.24	5.85	7.54	9.28	11.1	12.9	14.7	16.5	18.3
	32	0.34	1.27	2.52	3.95	5.49	7.13	8.83	10.6	12.4	14.1	16.0	17.8
	36	0.30	1.16	2.33	3.68	5.16	6.75	8.41	10.1	11.9	13.7	15.4	17.3

Table 7-9

Coefficients *C* for Eccentrically Loaded Bolt Groups

Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

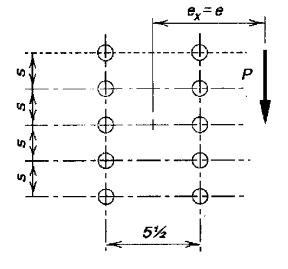
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	*e_x*, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.14	2.75	4.59	6.61	8.69	10.8	12.9	14.9	17.0	19.0	21.0	23.0
	3	0.94	2.32	3.92	5.80	7.82	9.90	12.0	14.1	16.2	18.3	20.4	22.4
	4	0.80	1.99	3.39	5.10	6.98	9.00	11.1	13.2	15.3	17.4	19.6	21.7
	5	0.70	1.74	2.96	4.51	6.24	8.15	10.2	12.3	14.4	16.5	18.6	20.8
	6	0.62	1.54	2.62	4.03	5.60	7.39	9.30	11.3	13.4	15.5	17.7	19.8
	7	0.55	1.38	2.36	3.63	5.07	6.72	8.53	10.5	12.5	14.6	16.7	18.8
	8	0.50	1.25	2.14	3.30	4.61	6.15	7.84	9.67	11.6	13.6	15.7	17.8
	9	0.46	1.14	1.96	3.01	4.22	5.66	7.23	8.97	10.8	12.8	14.8	16.9
	10	0.42	1.04	1.80	2.78	3.89	5.23	6.70	8.34	10.1	12.0	13.9	15.9
	12	0.37	0.90	1.55	2.39	3.36	4.53	5.82	7.28	8.87	10.6	12.4	14.3
	14	0.32	0.79	1.36	2.10	2.96	3.99	5.13	6.44	7.87	9.42	11.1	12.8
	16	0.29	0.70	1.21	1.87	2.64	3.55	4.58	5.76	7.05	8.47	9.99	11.6
	18	0.26	0.63	1.09	1.68	2.37	3.20	4.14	5.21	6.38	7.68	9.08	10.6
	20	0.24	0.57	0.99	1.53	2.16	2.91	3.77	4.75	5.82	7.02	8.30	9.69
	24	0.20	0.48	0.84	1.29	1.83	2.46	3.19	4.03	4.94	5.97	7.07	8.28
	28	0.18	0.42	0.73	1.11	1.58	2.13	2.77	3.49	4.29	5.19	6.15	7.21
	32	0.16	0.37	0.64	0.98	1.39	1.88	2.44	3.08	3.79	4.58	5.44	6.38
	36	0.14	0.33	0.57	0.88	1.24	1.68	2.18	2.75	3.39	4.10	4.87	5.72
	C′	5.40	12.3	21.2	32.3	45.8	61.8	80.3	102	125	152	181	213
6	2	1.14	3.25	5.37	7.45	9.49	11.5	13.5	15.5	17.5	19.5	21.4	23.4
	3	0.94	2.86	4.93	7.05	9.14	11.2	13.2	15.3	17.3	19.3	21.3	23.3
	4	0.80	2.52	4.47	6.59	8.72	10.8	12.9	15.0	17.0	19.0	21.0	23.1
	5	0.70	2.24	4.04	6.12	8.25	10.4	12.5	14.6	16.7	18.7	20.8	22.8
	6	0.62	2.00	3.65	5.66	7.77	9.91	12.1	14.2	16.3	18.4	20.4	22.5
	7	0.55	1.80	3.31	5.23	7.29	9.42	11.6	13.7	15.8	17.9	20.0	22.1
	8	0.50	1.64	3.02	4.84	6.83	8.93	11.1	13.2	15.4	17.5	19.6	21.7
	9	0.46	1.50	2.77	4.49	6.39	8.45	10.6	12.7	14.9	17.0	19.2	21.3
	10	0.42	1.38	2.56	4.18	5.99	7.99	10.1	12.2	14.4	16.5	18.7	20.8
	12	0.37	1.19	2.21	3.65	5.29	7.16	9.15	11.2	13.4	15.5	17.7	19.8
	14	0.32	1.04	1.95	3.24	4.72	6.44	8.32	10.3	12.4	14.5	16.7	18.8
	16	0.29	0.93	1.74	2.90	4.24	5.83	7.59	9.48	11.5	13.6	15.7	17.8
	18	0.26	0.84	1.57	2.62	3.84	5.31	6.95	8.74	10.7	12.6	14.7	16.8
	20	0.24	0.76	1.43	2.39	3.50	4.87	6.39	8.08	9.89	11.8	13.8	15.9
	24	0.20	0.64	1.21	2.02	2.98	4.16	5.49	6.99	8.61	10.4	12.2	14.1
	28	0.18	0.55	1.05	1.76	2.59	3.63	4.80	6.13	7.59	9.18	10.9	12.7
	32	0.16	0.49	0.93	1.55	2.29	3.21	4.25	5.45	6.77	8.21	9.76	11.4
	36	0.14	0.43	0.83	1.38	2.05	2.88	3.81	4.90	6.09	7.41	8.83	10.4
	C′	5.40	16.0	30.6	51.0	76.2	107	143	185	232	284	342	406

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

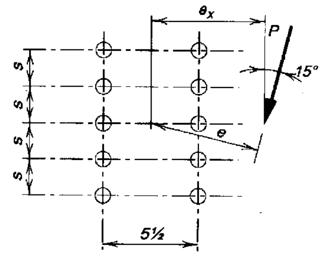
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.18	2.78	4.61	6.59	8.64	10.7	12.8	14.8	16.8	18.9	20.9	22.9
	3	0.97	2.34	3.97	5.80	7.78	9.83	11.9	14.0	16.1	18.1	20.2	22.2
	4	0.83	2.02	3.45	5.11	6.97	8.94	11.0	13.1	15.2	17.3	19.3	21.4
	5	0.72	1.77	3.03	4.54	6.26	8.12	10.1	12.1	14.2	16.3	18.4	20.5
	6	0.64	1.57	2.70	4.06	5.65	7.39	9.27	11.2	13.3	15.4	17.5	19.6
	7	0.57	1.41	2.43	3.66	5.13	6.74	8.52	10.4	12.4	14.4	16.5	18.6
	8	0.52	1.28	2.20	3.34	4.68	6.18	7.86	9.65	11.6	13.5	15.6	17.6
	9	0.48	1.17	2.01	3.06	4.30	5.70	7.27	8.97	10.8	12.7	14.7	16.7
	10	0.44	1.07	1.85	2.82	3.98	5.27	6.76	8.36	10.1	11.9	13.8	15.8
	12	0.38	0.93	1.60	2.44	3.44	4.58	5.90	7.34	8.91	10.6	12.4	14.2
	14	0.33	0.81	1.40	2.15	3.03	4.05	5.22	6.51	7.94	9.47	11.1	12.8
	16	0.30	0.72	1.25	1.91	2.70	3.62	4.68	5.84	7.14	8.54	10.1	11.7
	18	0.27	0.65	1.13	1.72	2.44	3.27	4.23	5.28	6.48	7.77	9.16	10.7
	20	0.25	0.59	1.02	1.57	2.22	2.98	3.86	4.83	5.93	7.11	8.40	9.78
	24	0.21	0.50	0.87	1.33	1.88	2.53	3.27	4.11	5.05	6.07	7.19	8.39
	28	0.18	0.43	0.75	1.15	1.63	2.19	2.84	3.57	4.39	5.29	6.28	7.33
	32	0.16	0.38	0.66	1.01	1.43	1.93	2.50	3.15	3.88	4.68	5.56	6.50
	36	0.14	0.34	0.59	0.90	1.28	1.73	2.24	2.82	3.48	4.19	4.99	5.84
6	2	1.18	3.24	5.34	7.40	9.43	11.5	13.5	15.4	17.4	19.4	21.4	23.4
	3	0.97	2.85	4.90	6.99	9.07	11.1	13.2	15.2	17.2	19.2	21.2	23.2
	4	0.83	2.51	4.45	6.53	8.63	10.7	12.8	14.8	16.9	18.9	20.9	22.9
	5	0.72	2.23	4.05	6.07	8.16	10.3	12.4	14.5	16.5	18.6	20.6	22.6
	6	0.64	2.00	3.68	5.62	7.69	9.80	11.9	14.0	16.1	18.2	20.2	22.3
	7	0.57	1.81	3.36	5.20	7.22	9.31	11.4	13.5	15.7	17.7	19.8	21.9
	8	0.52	1.65	3.08	4.82	6.78	8.83	10.9	13.1	15.2	17.3	19.4	21.5
	9	0.48	1.52	2.83	4.48	6.36	8.37	10.5	12.6	14.7	16.8	18.9	21.0
	10	0.44	1.40	2.62	4.18	5.98	7.93	9.97	12.1	14.2	16.3	18.4	20.6
	12	0.38	1.21	2.27	3.66	5.31	7.13	9.08	11.1	13.2	15.3	17.4	19.6
	14	0.33	1.07	2.00	3.25	4.76	6.44	8.28	10.2	12.3	14.3	16.4	18.6
	16	0.30	0.95	1.79	2.92	4.29	5.85	7.58	9.43	11.4	13.4	15.5	17.6
	18	0.27	0.86	1.62	2.65	3.90	5.34	6.97	8.72	10.6	12.5	14.6	16.6
	20	0.25	0.78	1.47	2.42	3.58	4.91	6.43	8.09	9.87	11.7	13.7	15.7
	24	0.21	0.66	1.25	2.06	3.05	4.21	5.55	7.03	8.64	10.4	12.2	14.1
	28	0.18	0.57	1.08	1.79	2.66	3.68	4.87	6.19	7.65	9.22	10.9	12.6
	32	0.16	0.50	0.95	1.58	2.35	3.26	4.33	5.52	6.84	8.27	9.81	11.4
	36	0.14	0.45	0.85	1.42	2.11	2.93	3.90	4.97	6.18	7.49	8.91	10.4

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

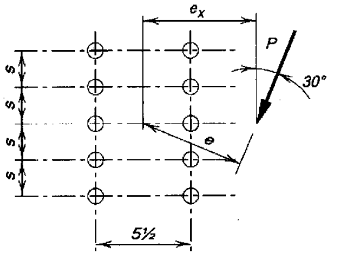
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.30	2.90	4.72	6.66	8.65	10.7	12.7	14.7	16.7	18.7	20.8	22.8
	3	1.08	2.47	4.13	5.94	7.86	9.85	11.9	13.9	16.0	18.0	20.0	22.1
	4	0.92	2.14	3.64	5.30	7.12	9.04	11.0	13.0	15.1	17.1	19.2	21.2
	5	0.80	1.89	3.24	4.76	6.46	8.29	10.2	12.2	14.2	16.3	18.3	20.4
	6	0.71	1.69	2.91	4.29	5.88	7.61	9.45	11.4	13.4	15.4	17.4	19.5
	7	0.64	1.53	2.63	3.90	5.38	7.01	8.76	10.6	12.5	14.5	16.5	18.6
	8	0.58	1.39	2.40	3.57	4.95	6.49	8.14	9.92	11.8	13.7	15.7	17.7
	9	0.53	1.28	2.20	3.29	4.58	6.02	7.59	9.29	11.1	12.9	14.9	16.8
	10	0.49	1.18	2.03	3.04	4.26	5.61	7.09	8.72	10.4	12.2	14.1	16.0
	12	0.42	1.02	1.76	2.65	3.72	4.92	6.25	7.73	9.31	11.0	12.8	14.6
	14	0.37	0.90	1.55	2.34	3.29	4.37	5.58	6.93	8.38	9.93	11.6	13.3
	16	0.33	0.80	1.38	2.09	2.95	3.92	5.03	6.26	7.59	9.03	10.6	12.2
	18	0.30	0.72	1.25	1.89	2.67	3.55	4.57	5.70	6.93	8.27	9.70	11.2
	20	0.27	0.66	1.13	1.73	2.43	3.25	4.19	5.23	6.36	7.62	8.95	10.4
	24	0.23	0.56	0.96	1.46	2.07	2.77	3.57	4.47	5.47	6.56	7.73	8.99
	28	0.20	0.48	0.83	1.27	1.79	2.41	3.11	3.90	4.78	5.75	6.78	7.91
	32	0.18	0.43	0.73	1.12	1.58	2.13	2.76	3.46	4.25	5.11	6.04	7.06
	36	0.16	0.38	0.66	1.00	1.42	1.91	2.47	3.10	3.81	4.59	5.44	6.36
6	2	1.30	3.27	5.33	7.36	9.38	11.4	13.4	15.4	17.4	19.3	21.3	23.3
	3	1.08	2.89	4.91	6.96	9.01	11.0	13.1	15.1	17.1	19.1	21.1	23.1
	4	0.92	2.56	4.50	6.53	8.58	10.6	12.7	14.7	16.8	18.8	20.8	22.8
	5	0.80	2.29	4.13	6.10	8.14	10.2	12.3	14.3	16.4	18.4	20.4	22.5
	6	0.71	2.08	3.80	5.69	7.70	9.75	11.8	13.9	15.9	18.0	20.0	22.1
	7	0.64	1.89	3.51	5.31	7.27	9.30	11.4	13.4	15.5	17.6	19.6	21.7
	8	0.58	1.74	3.25	4.96	6.86	8.86	10.9	13.0	15.0	17.1	19.2	21.3
	9	0.53	1.61	3.02	4.64	6.49	8.44	10.5	12.5	14.6	16.7	18.7	20.8
	10	0.49	1.49	2.81	4.35	6.13	8.04	10.0	12.1	14.1	16.2	18.3	20.4
	12	0.42	1.30	2.47	3.85	5.51	7.31	9.22	11.2	13.2	15.3	17.3	19.4
	14	0.37	1.15	2.19	3.44	4.98	6.67	8.49	10.4	12.4	14.4	16.4	18.5
	16	0.33	1.03	1.96	3.11	4.54	6.12	7.83	9.66	11.6	13.5	15.6	17.6
	18	0.30	0.93	1.78	2.83	4.16	5.63	7.26	9.00	10.8	12.8	14.7	16.7
	20	0.27	0.85	1.62	2.60	3.83	5.21	6.74	8.41	10.2	12.0	13.9	15.9
	24	0.23	0.72	1.38	2.23	3.30	4.51	5.89	7.40	9.02	10.7	12.5	14.4
	28	0.20	0.63	1.20	1.95	2.89	3.96	5.21	6.59	8.07	9.66	11.3	13.1
	32	0.18	0.55	1.06	1.73	2.57	3.53	4.67	5.92	7.28	8.75	10.3	12.0
	36	0.16	0.50	0.95	1.55	2.31	3.18	4.22	5.36	6.61	7.98	9.43	11.0

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

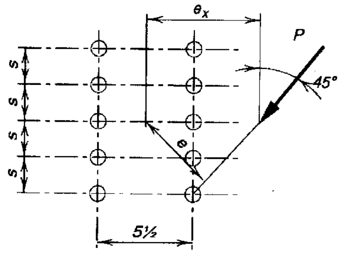
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.53	3.18	4.96	6.84	8.77	10.7	12.7	14.7	16.7	18.7	20.7	22.6
	3	1.30	2.76	4.42	6.22	8.09	10.0	12.0	14.0	15.9	17.9	19.9	21.9
	4	1.11	2.43	3.97	5.67	7.46	9.32	11.2	13.2	15.2	17.2	19.2	21.2
	5	0.98	2.17	3.60	5.19	6.89	8.68	10.6	12.5	14.4	16.4	18.4	20.4
	6	0.87	1.95	3.28	4.77	6.37	8.09	9.90	11.8	13.7	15.6	17.6	19.6
	7	0.78	1.78	3.01	4.40	5.91	7.56	9.31	11.1	13.0	14.9	16.9	18.8
	8	0.71	1.63	2.77	4.07	5.50	7.07	8.76	10.5	12.4	14.2	16.2	18.1
	9	0.65	1.50	2.57	3.78	5.13	6.64	8.26	9.97	11.8	13.6	15.5	17.4
	10	0.60	1.39	2.39	3.52	4.81	6.25	7.81	9.45	11.2	13.0	14.8	16.7
	12	0.52	1.22	2.08	3.09	4.26	5.58	7.01	8.54	10.2	11.9	13.6	15.4
	14	0.45	1.08	1.85	2.75	3.82	5.02	6.34	7.76	9.28	10.9	12.6	14.3
	16	0.41	0.96	1.65	2.48	3.45	4.55	5.77	7.09	8.53	10.1	11.6	13.3
	18	0.37	0.87	1.50	2.25	3.14	4.16	5.29	6.53	7.87	9.30	10.8	12.4
	20	0.33	0.79	1.37	2.06	2.88	3.82	4.87	6.04	7.30	8.65	10.1	11.6
	24	0.28	0.68	1.16	1.76	2.47	3.28	4.21	5.23	6.35	7.55	8.85	10.2
	28	0.25	0.59	1.01	1.53	2.15	2.87	3.69	4.61	5.61	6.69	7.87	9.11
	32	0.22	0.52	0.89	1.35	1.91	2.55	3.29	4.11	5.01	6.00	7.07	8.20
	36	0.20	0.46	0.80	1.21	1.71	2.29	2.96	3.70	4.53	5.43	6.40	7.44
6	2	1.53	3.39	5.36	7.35	9.35	11.3	13.3	15.3	17.3	19.3	21.3	23.2
	3	1.30	3.04	4.99	6.98	8.98	11.0	13.0	15.0	17.0	19.0	21.0	22.9
	4	1.11	2.74	4.64	6.60	8.60	10.6	12.6	14.6	16.6	18.6	20.6	22.6
	5	0.98	2.49	4.31	6.24	8.21	10.2	12.2	14.2	16.3	18.3	20.3	22.3
	6	0.87	2.28	4.02	5.89	7.84	9.82	11.8	13.8	15.9	17.9	19.9	21.9
	7	0.78	2.10	3.76	5.57	7.48	9.44	11.4	13.4	15.5	17.5	19.5	21.5
	8	0.71	1.94	3.53	5.28	7.13	9.07	11.0	13.0	15.1	17.1	19.1	21.1
	9	0.65	1.81	3.32	5.00	6.81	8.71	10.7	12.7	14.7	16.7	18.7	20.7
	10	0.60	1.69	3.13	4.74	6.50	8.37	10.3	12.3	14.3	16.3	18.3	20.3
	12	0.52	1.50	2.80	4.29	5.94	7.74	9.61	11.5	13.5	15.5	17.5	19.5
	14	0.45	1.34	2.52	3.89	5.45	7.17	8.98	10.9	12.8	14.7	16.7	18.7
	16	0.41	1.21	2.29	3.55	5.02	6.67	8.41	10.2	12.1	14.0	16.0	17.9
	18	0.37	1.10	2.09	3.26	4.65	6.22	7.89	9.65	11.5	13.4	15.3	17.2
	20	0.33	1.01	1.92	3.01	4.33	5.82	7.42	9.11	10.9	12.7	14.6	16.5
	24	0.28	0.86	1.64	2.61	3.79	5.13	6.60	8.17	9.84	11.6	13.4	15.2
	28	0.25	0.75	1.44	2.30	3.36	4.58	5.92	7.38	8.95	10.6	12.3	14.1
	32	0.22	0.67	1.27	2.05	3.02	4.12	5.35	6.72	8.18	9.73	11.4	13.0
	36	0.20	0.60	1.14	1.85	2.73	3.74	4.88	6.15	7.52	8.98	10.5	12.1

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

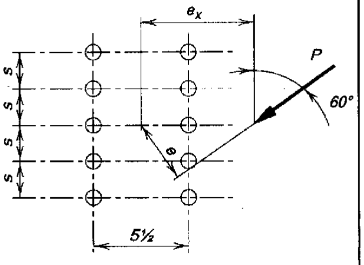
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.78	3.55	5.34	7.17	9.04	10.9	12.9	14.8	16.7	18.7	20.6	22.6
	3	1.62	3.26	4.95	6.71	8.53	10.4	12.3	14.2	16.1	18.1	20.0	22.0
	4	1.45	2.97	4.57	6.27	8.04	9.86	11.7	13.6	15.5	17.5	19.4	21.4
	5	1.31	2.71	4.23	5.86	7.58	9.36	11.2	13.1	15.0	16.9	18.8	20.7
	6	1.18	2.48	3.93	5.50	7.16	8.90	10.7	12.5	14.4	16.3	18.2	20.1
	7	1.07	2.28	3.66	5.18	6.79	8.48	10.2	12.0	13.9	15.7	17.6	19.5
	8	0.98	2.11	3.43	4.88	6.45	8.09	9.80	11.6	13.4	15.2	17.1	19.0
	9	0.90	1.97	3.22	4.61	6.12	7.72	9.39	11.1	12.9	14.7	16.6	18.4
	10	0.83	1.84	3.03	4.37	5.82	7.37	9.00	10.7	12.5	14.2	16.1	17.9
	12	0.72	1.62	2.70	3.93	5.28	6.73	8.28	9.91	11.6	13.4	15.1	16.9
	14	0.64	1.45	2.43	3.56	4.81	6.19	7.66	9.22	10.9	12.5	14.3	16.0
	16	0.57	1.31	2.21	3.24	4.42	5.71	7.11	8.60	10.2	11.8	13.5	15.2
	18	0.52	1.19	2.02	2.98	4.07	5.29	6.63	8.05	9.55	11.1	12.7	14.4
	20	0.47	1.09	1.85	2.75	3.77	4.93	6.19	7.55	8.98	10.5	12.1	13.7
	24	0.40	0.93	1.59	2.37	3.28	4.32	5.46	6.69	8.01	9.41	10.9	12.4
	28	0.35	0.82	1.39	2.08	2.90	3.83	4.86	5.99	7.21	8.51	9.88	11.3
	32	0.31	0.72	1.24	1.86	2.59	3.43	4.37	5.41	6.54	7.75	9.02	10.4
	36	0.28	0.65	1.11	1.67	2.34	3.11	3.97	4.93	5.98	7.10	8.29	9.55
6	2	1.78	3.59	5.48	7.41	9.36	11.3	13.3	15.3	17.2	19.2	21.2	23.2
	3	1.62	3.35	5.20	7.12	9.06	11.0	13.0	15.0	16.9	18.9	20.9	22.9
	4	1.45	3.11	4.93	6.82	8.75	10.7	12.7	14.6	16.6	18.6	20.6	22.5
	5	1.31	2.89	4.66	6.53	8.45	10.4	12.3	14.3	16.3	18.2	20.2	22.2
	6	1.18	2.70	4.42	6.26	8.16	10.1	12.0	14.0	15.9	17.9	19.9	21.9
	7	1.07	2.52	4.19	6.01	7.88	9.79	11.7	13.7	15.6	17.6	19.6	21.5
	8	0.98	2.36	3.99	5.77	7.62	9.51	11.4	13.4	15.3	17.3	19.2	21.2
	9	0.90	2.23	3.81	5.55	7.37	9.24	11.1	13.1	15.0	17.0	18.9	20.9
	10	0.83	2.10	3.64	5.35	7.13	8.98	10.9	12.8	14.7	16.7	18.6	20.6
	12	0.72	1.89	3.34	4.97	6.70	8.49	10.3	12.2	14.1	16.1	18.0	19.9
	14	0.64	1.71	3.08	4.63	6.29	8.04	9.85	11.7	13.6	15.5	17.4	19.3
	16	0.57	1.57	2.85	4.32	5.92	7.62	9.39	11.2	13.1	15.0	16.9	18.8
	18	0.52	1.44	2.65	4.04	5.58	7.22	8.95	10.7	12.6	14.4	16.3	18.2
	20	0.47	1.33	2.47	3.79	5.26	6.86	8.55	10.3	12.1	13.9	15.8	17.7
	24	0.40	1.16	2.17	3.36	4.71	6.21	7.82	9.50	11.2	13.0	14.8	16.7
	28	0.35	1.02	1.92	3.00	4.26	5.67	7.19	8.80	10.5	12.2	14.0	15.8
	32	0.31	0.91	1.72	2.71	3.88	5.20	6.64	8.17	9.77	11.4	13.1	14.9
	36	0.28	0.82	1.56	2.46	3.55	4.80	6.16	7.61	9.14	10.7	12.4	14.1

Table 7-9 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

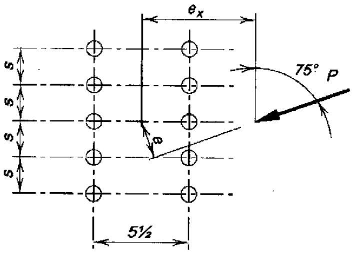
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.92	3.82	5.70	7.57	9.45	11.3	13.2	15.2	17.1	19.0	20.9	22.9
	3	1.87	3.72	5.54	7.36	9.19	11.1	12.9	14.8	16.7	18.6	20.5	22.5
	4	1.82	3.60	5.37	7.14	8.94	10.8	12.6	14.5	16.3	18.2	20.1	22.1
	5	1.75	3.47	5.18	6.92	8.68	10.5	12.3	14.1	16.0	17.9	19.8	21.7
	6	1.68	3.33	5.00	6.69	8.42	10.2	12.0	13.8	15.7	17.5	19.4	21.3
	7	1.60	3.19	4.81	6.47	8.17	9.92	11.7	13.5	15.3	17.2	19.1	20.9
	8	1.52	3.06	4.63	6.26	7.93	9.66	11.4	13.2	15.0	16.9	18.7	20.6
	9	1.45	2.93	4.46	6.05	7.70	9.41	11.2	12.9	14.7	16.5	18.4	20.3
	10	1.38	2.80	4.29	5.85	7.48	9.16	10.9	12.6	14.4	16.2	18.1	19.9
	12	1.25	2.57	3.98	5.48	7.07	8.71	10.4	12.1	13.9	15.7	17.5	19.3
	14	1.13	2.36	3.70	5.15	6.69	8.29	9.96	11.7	13.4	15.2	16.9	18.7
	16	1.03	2.18	3.45	4.85	6.34	7.90	9.53	11.2	12.9	14.7	16.4	18.2
	18	0.95	2.02	3.23	4.57	6.01	7.54	9.13	10.8	12.5	14.2	15.9	17.7
	20	0.87	1.88	3.03	4.32	5.71	7.19	8.75	10.4	12.0	13.7	15.4	17.2
	24	0.75	1.65	2.69	3.87	5.17	6.57	8.05	9.60	11.2	12.9	14.5	16.2
	28	0.66	1.46	2.42	3.50	4.71	6.03	7.44	8.93	10.5	12.1	13.7	15.4
	32	0.59	1.31	2.18	3.19	4.32	5.56	6.90	8.32	9.81	11.4	12.9	14.6
	36	0.53	1.19	1.99	2.92	3.98	5.15	6.42	7.78	9.21	10.7	12.2	13.8
6	2	1.92	3.80	5.69	7.59	9.51	11.5	13.4	15.4	17.6	19.6	21.5	23.5
	3	1.87	3.70	5.55	7.42	9.32	11.2	13.2	15.1	17.1	19.0	21.0	23.0
	4	1.82	3.59	5.40	7.25	9.14	11.1	13.0	14.9	16.9	18.8	20.8	22.7
	5	1.75	3.48	5.26	7.09	8.96	10.9	12.8	14.7	16.6	18.6	20.5	22.5
	6	1.68	3.36	5.11	6.93	8.78	10.7	12.6	14.5	16.4	18.4	20.3	22.2
	7	1.60	3.24	4.97	6.77	8.62	10.5	12.4	14.3	16.2	18.1	20.1	22.0
	8	1.52	3.13	4.84	6.62	8.45	10.3	12.2	14.1	16.0	17.9	19.9	21.8
	9	1.45	3.02	4.71	6.47	8.29	10.2	12.0	13.9	15.8	17.7	19.7	21.6
	10	1.38	2.91	4.58	6.33	8.14	9.98	11.9	13.7	15.6	17.6	19.5	21.4
	12	1.25	2.72	4.34	6.07	7.85	9.67	11.5	13.4	15.3	17.2	19.1	21.0
	14	1.13	2.54	4.13	5.82	7.57	9.38	11.2	13.1	15.0	16.8	18.7	20.6
	16	1.03	2.38	3.92	5.59	7.32	9.10	10.9	12.8	14.6	16.5	18.4	20.3
	18	0.95	2.24	3.74	5.38	7.09	8.85	10.7	12.5	14.3	16.2	18.1	19.9
	20	0.87	2.11	3.57	5.17	6.87	8.61	10.4	12.2	14.0	15.9	17.7	19.6
	24	0.75	1.88	3.27	4.80	6.44	8.15	9.90	11.7	13.5	15.3	17.1	19.0
	28	0.66	1.70	3.00	4.47	6.06	7.72	9.43	11.2	13.0	14.8	16.6	18.4
	32	0.59	1.55	2.77	4.17	5.70	7.31	8.99	10.7	12.5	14.3	16.1	17.9
	36	0.53	1.42	2.57	3.90	5.37	6.93	8.57	10.3	12.0	13.8	15.5	17.3

Table 7–10
Coefficients C for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\quad\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

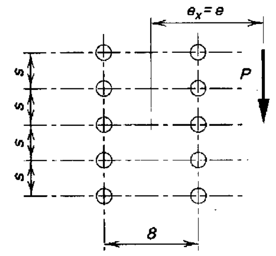
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.31	2.91	4.71	6.66	8.69	10.8	12.8	14.9	16.9	18.9	21.0	23.0
	3	1.12	2.54	4.14	5.95	7.90	9.93	12.0	14.1	16.2	18.2	20.3	22.4
	4	0.98	2.24	3.66	5.33	7.15	9.10	11.1	13.2	15.3	17.4	19.5	21.6
	5	0.87	1.99	3.27	4.80	6.48	8.33	10.3	12.3	14.4	16.5	18.6	20.7
	6	0.79	1.80	2.95	4.35	5.90	7.63	9.49	11.5	13.5	15.6	17.7	19.8
	7	0.71	1.63	2.68	3.97	5.40	7.02	8.77	10.7	12.6	14.6	16.7	18.8
	8	0.65	1.49	2.46	3.65	4.97	6.48	8.13	9.91	11.8	13.8	15.8	17.9
	9	0.60	1.38	2.27	3.37	4.59	6.01	7.55	9.24	11.1	13.0	14.9	17.0
	10	0.56	1.28	2.11	3.13	4.27	5.59	7.04	8.64	10.4	12.2	14.1	16.1
	12	0.49	1.11	1.84	2.73	3.73	4.90	6.19	7.63	9.18	10.9	12.6	14.5
	14	0.44	0.99	1.64	2.42	3.31	4.36	5.50	6.80	8.20	9.73	11.4	13.1
	16	0.39	0.89	1.47	2.17	2.98	3.91	4.95	6.13	7.40	8.80	10.3	11.9
	18	0.36	0.80	1.33	1.97	2.70	3.55	4.50	5.57	6.73	8.02	9.39	10.9
	20	0.33	0.73	1.22	1.80	2.47	3.25	4.12	5.10	6.17	7.35	8.62	9.99
	24	0.28	0.63	1.04	1.53	2.10	2.77	3.51	4.35	5.28	6.30	7.39	8.59
	28	0.25	0.55	0.91	1.33	1.83	2.41	3.06	3.79	4.60	5.50	6.46	7.51
	32	0.22	0.48	0.80	1.18	1.62	2.13	2.71	3.36	4.08	4.87	5.73	6.67
	36	0.20	0.43	0.72	1.06	1.45	1.91	2.43	3.01	3.66	4.37	5.15	5.99
	C′	7.85	16.8	27.3	39.9	54.6	71.5	90.9	113	137	164	194	226
6	2	1.31	3.28	5.35	7.42	9.47	11.5	13.5	15.5	17.5	19.5	21.4	23.4
	3	1.12	2.93	4.94	7.03	9.12	11.2	13.2	15.3	17.3	19.3	21.3	23.3
	4	0.98	2.63	4.52	6.59	8.70	10.8	12.9	14.9	17.0	19.0	21.0	23.0
	5	0.87	2.37	4.13	6.15	8.25	10.4	12.5	14.6	16.6	18.7	20.7	22.8
	6	0.79	2.15	3.78	5.72	7.78	9.90	12.0	14.1	16.2	18.3	20.4	22.4
	7	0.71	1.97	3.47	5.32	7.33	9.43	11.6	13.7	15.8	17.9	20.0	22.1
	8	0.65	1.81	3.19	4.95	6.89	8.95	11.1	13.2	15.4	17.5	19.6	21.7
	9	0.60	1.67	2.95	4.62	6.48	8.49	10.6	12.7	14.9	17.0	19.1	21.3
	10	0.56	1.55	2.75	4.33	6.10	8.05	10.1	12.2	14.4	16.5	18.7	20.8
	12	0.49	1.35	2.40	3.82	5.43	7.25	9.21	11.3	13.4	15.5	17.7	19.8
	14	0.44	1.20	2.14	3.41	4.86	6.56	8.40	10.4	12.4	14.5	16.7	18.8
	16	0.39	1.08	1.92	3.07	4.40	5.96	7.69	9.56	11.5	13.6	15.7	17.8
	18	0.36	0.97	1.75	2.79	4.00	5.46	7.06	8.83	10.7	12.7	14.7	16.8
	20	0.33	0.89	1.60	2.56	3.67	5.02	6.52	8.18	9.97	11.9	13.9	15.9
	24	0.28	0.76	1.37	2.18	3.14	4.32	5.62	7.11	8.71	10.4	12.3	14.2
	28	0.25	0.66	1.19	1.90	2.75	3.78	4.93	6.26	7.70	9.27	11.0	12.7
	32	0.22	0.58	1.05	1.68	2.44	3.35	4.38	5.58	6.88	8.31	9.85	11.5
	36	0.20	0.52	0.95	1.51	2.19	3.01	3.94	5.02	6.21	7.52	8.93	10.4
	C′	7.85	19.6	35.6	56.6	82.5	114	150	192	239	292	350	414

Table 7-10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

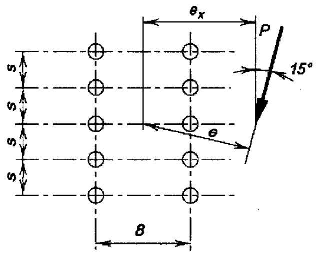
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.35	2.96	4.75	6.67	8.67	10.7	12.7	14.8	16.8	18.8	20.9	22.9
	3	1.16	2.58	4.20	5.98	7.90	9.89	11.9	14.0	16.0	18.1	20.2	22.2
	4	1.02	2.28	3.73	5.37	7.17	9.08	11.1	13.1	15.2	17.3	19.3	21.4
	5	0.90	2.03	3.35	4.85	6.53	8.34	10.3	12.2	14.3	16.3	18.4	20.5
	6	0.81	1.84	3.03	4.40	5.96	7.66	9.48	11.4	13.4	15.4	17.5	19.6
	7	0.74	1.67	2.76	4.02	5.48	7.06	8.79	10.6	12.6	14.5	16.6	18.6
	8	0.68	1.53	2.53	3.70	5.05	6.53	8.17	9.91	11.8	13.7	15.7	17.7
	9	0.63	1.42	2.34	3.43	4.68	6.07	7.61	9.27	11.0	12.9	14.8	16.8
	10	0.58	1.31	2.17	3.19	4.36	5.66	7.12	8.69	10.4	12.2	14.0	16.0
	12	0.51	1.15	1.90	2.79	3.82	4.97	6.28	7.69	9.23	10.9	12.6	14.4
	14	0.45	1.02	1.69	2.48	3.40	4.43	5.61	6.88	8.29	9.79	11.4	13.1
	16	0.41	0.91	1.51	2.23	3.05	3.99	5.05	6.21	7.50	8.88	10.4	11.9
	18	0.37	0.83	1.37	2.02	2.77	3.63	4.60	5.66	6.84	8.11	9.48	11.0
	20	0.34	0.76	1.26	1.85	2.54	3.32	4.21	5.19	6.28	7.45	8.73	10.1
	24	0.29	0.65	1.07	1.58	2.16	2.84	3.60	4.45	5.39	6.40	7.52	8.71
	28	0.25	0.56	0.93	1.37	1.89	2.47	3.14	3.88	4.71	5.61	6.59	7.64
	32	0.23	0.50	0.83	1.22	1.67	2.19	2.78	3.44	4.18	4.98	5.86	6.80
	36	0.20	0.45	0.74	1.09	1.50	1.96	2.49	3.09	3.75	4.47	5.27	6.12
6	2	1.35	3.29	5.33	7.39	9.42	11.4	13.4	15.4	17.4	19.4	21.4	23.4
	3	1.16	2.94	4.93	6.99	9.05	11.1	13.1	15.2	17.2	19.2	21.2	23.2
	4	1.02	2.64	4.52	6.55	8.63	10.7	12.8	14.8	16.9	18.9	20.9	22.9
	5	0.90	2.38	4.15	6.12	8.18	10.3	12.4	14.4	16.5	18.5	20.6	22.6
	6	0.81	2.17	3.82	5.70	7.72	9.80	11.9	14.0	16.1	18.2	20.2	22.3
	7	0.74	1.99	3.52	5.31	7.28	9.33	11.4	13.5	15.6	17.7	19.8	21.9
	8	0.68	1.83	3.25	4.95	6.86	8.87	11.0	13.1	15.2	17.3	19.4	21.5
	9	0.63	1.69	3.02	4.63	6.46	8.43	10.5	12.6	14.7	16.8	18.9	21.0
	10	0.58	1.58	2.81	4.34	6.10	8.00	10.0	12.1	14.2	16.3	18.4	20.5
	12	0.51	1.38	2.47	3.84	5.45	7.23	9.15	11.2	13.2	15.3	17.4	19.6
	14	0.45	1.23	2.20	3.44	4.91	6.56	8.38	10.3	12.3	14.4	16.5	18.6
	16	0.41	1.10	1.98	3.11	4.46	5.99	7.69	9.52	11.5	13.5	15.5	17.6
	18	0.37	1.00	1.80	2.83	4.08	5.49	7.09	8.82	10.7	12.6	14.6	16.6
	20	0.34	0.92	1.65	2.60	3.75	5.06	6.56	8.20	9.96	11.8	13.8	15.7
	24	0.29	0.78	1.41	2.23	3.22	4.36	5.70	7.15	8.74	10.4	12.2	14.1
	28	0.25	0.68	1.23	1.95	2.82	3.83	5.02	6.32	7.76	9.31	11.0	12.7
	32	0.23	0.60	1.09	1.73	2.50	3.41	4.47	5.64	6.96	8.38	9.90	11.5
	36	0.20	0.54	0.97	1.55	2.25	3.07	4.03	5.09	6.30	7.60	9.01	10.5

Table 7–10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

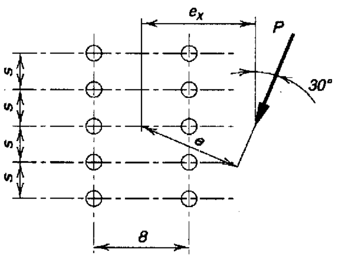
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.49	3.12	4.91	6.80	8.75	10.7	12.7	14.7	16.7	18.7	20.8	22.7
	3	1.29	2.74	4.39	6.16	8.04	9.98	12.0	14.0	16.0	18.0	20.0	22.1
	4	1.13	2.43	3.95	5.60	7.37	9.24	11.2	13.2	15.2	17.2	19.2	21.3
	5	1.00	2.18	3.58	5.10	6.77	8.55	10.4	12.4	14.3	16.3	18.4	20.4
	6	0.90	1.98	3.26	4.67	6.23	7.93	9.72	11.6	13.5	15.5	17.5	19.5
	7	0.82	1.81	2.99	4.30	5.76	7.37	9.08	10.9	12.8	14.7	16.7	18.7
	8	0.75	1.67	2.76	3.97	5.35	6.87	8.49	10.2	12.0	13.9	15.9	17.8
	9	0.70	1.55	2.56	3.69	4.98	6.42	7.96	9.62	11.4	13.2	15.1	17.0
	10	0.65	1.44	2.38	3.44	4.66	6.02	7.49	9.07	10.8	12.5	14.4	16.2
	12	0.57	1.26	2.09	3.03	4.13	5.34	6.66	8.12	9.67	11.3	13.0	14.8
	14	0.50	1.12	1.86	2.71	3.69	4.78	5.99	7.33	8.75	10.3	11.9	13.6
	16	0.45	1.01	1.67	2.44	3.33	4.33	5.44	6.66	7.98	9.39	10.9	12.5
	18	0.41	0.92	1.52	2.22	3.03	3.95	4.97	6.10	7.32	8.64	10.1	11.5
	20	0.38	0.84	1.39	2.03	2.78	3.62	4.57	5.62	6.75	7.98	9.30	10.7
	24	0.32	0.72	1.19	1.74	2.38	3.11	3.93	4.84	5.83	6.92	8.08	9.32
	28	0.28	0.63	1.04	1.52	2.08	2.72	3.44	4.24	5.13	6.09	7.12	8.24
	32	0.25	0.56	0.92	1.35	1.84	2.41	3.06	3.77	4.57	5.43	6.36	7.37
	36	0.23	0.50	0.83	1.21	1.66	2.17	2.75	3.40	4.11	4.89	5.74	6.66
6	2	1.49	3.36	5.36	7.37	9.38	11.4	13.4	15.4	17.4	19.3	21.3	23.3
	3	1.29	3.02	4.97	6.99	9.01	11.0	13.1	15.1	17.1	19.1	21.1	23.1
	4	1.13	2.73	4.60	6.58	8.61	10.7	12.7	14.7	16.7	18.8	20.8	22.8
	5	1.00	2.48	4.26	6.18	8.18	10.2	12.3	14.3	16.4	18.4	20.4	22.4
	6	0.90	2.27	3.96	5.80	7.76	9.79	11.8	13.9	15.9	18.0	20.0	22.1
	7	0.82	2.09	3.68	5.44	7.36	9.35	11.4	13.5	15.5	17.6	19.6	21.7
	8	0.75	1.93	3.43	5.11	6.97	8.93	11.0	13.0	15.1	17.1	19.2	21.2
	9	0.70	1.80	3.21	4.81	6.61	8.53	10.5	12.6	14.6	16.7	18.7	20.8
	10	0.65	1.68	3.01	4.53	6.27	8.14	10.1	12.1	14.2	16.2	18.3	20.4
	12	0.57	1.49	2.67	4.05	5.67	7.43	9.31	11.3	13.3	15.3	17.4	19.4
	14	0.50	1.33	2.39	3.65	5.15	6.81	8.60	10.5	12.4	14.4	16.5	18.5
	16	0.45	1.20	2.16	3.31	4.71	6.27	7.96	9.76	11.7	13.6	15.6	17.6
	18	0.41	1.09	1.97	3.03	4.34	5.79	7.39	9.12	10.9	12.8	14.8	16.8
	20	0.38	1.00	1.81	2.80	4.01	5.37	6.89	8.53	10.3	12.1	14.0	15.9
	24	0.32	0.86	1.55	2.41	3.48	4.68	6.04	7.53	9.14	10.8	12.6	14.5
	28	0.28	0.75	1.35	2.12	3.06	4.13	5.36	6.72	8.19	9.76	11.4	13.2
	32	0.25	0.67	1.20	1.89	2.73	3.69	4.81	6.05	7.40	8.86	10.4	12.0
	36	0.23	0.60	1.08	1.70	2.46	3.34	4.36	5.50	6.74	8.09	9.53	11.1

Table 7-10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

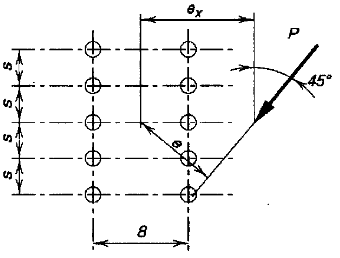
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.70	3.43	5.22	7.06	8.95	10.9	12.8	14.8	16.8	18.7	20.7	22.7
	3	1.51	3.09	4.76	6.52	8.35	10.2	12.2	14.1	16.1	18.0	20.0	22.0
	4	1.35	2.78	4.34	6.01	7.78	9.60	11.5	13.4	15.3	17.3	19.3	21.3
	5	1.21	2.52	3.97	5.57	7.25	9.01	10.8	12.7	14.6	16.6	18.5	20.5
	6	1.10	2.30	3.67	5.17	6.78	8.47	10.2	12.1	13.9	15.9	17.8	19.8
	7	1.00	2.12	3.40	4.82	6.35	7.97	9.67	11.5	13.3	15.2	17.1	19.0
	8	0.92	1.96	3.17	4.51	5.96	7.51	9.15	10.9	12.7	14.5	16.4	18.3
	9	0.85	1.82	2.96	4.23	5.60	7.08	8.68	10.4	12.1	13.9	15.7	17.6
	10	0.79	1.70	2.78	3.97	5.28	6.70	8.24	9.86	11.5	13.3	15.1	17.0
	12	0.69	1.50	2.46	3.54	4.73	6.04	7.46	8.97	10.6	12.2	14.0	15.7
	14	0.61	1.34	2.21	3.18	4.27	5.48	6.80	8.21	9.70	11.3	12.9	14.6
	16	0.55	1.21	2.00	2.88	3.89	5.01	6.23	7.54	8.95	10.4	12.0	13.6
	18	0.50	1.11	1.82	2.64	3.56	4.60	5.74	6.97	8.30	9.71	11.2	12.7
	20	0.46	1.02	1.67	2.42	3.29	4.25	5.31	6.47	7.73	9.06	10.5	11.9
	24	0.40	0.87	1.43	2.09	2.84	3.68	4.62	5.65	6.77	7.96	9.23	10.6
	28	0.35	0.76	1.26	1.83	2.49	3.24	4.07	5.00	6.00	7.08	8.24	9.47
	32	0.31	0.68	1.12	1.63	2.22	2.89	3.64	4.47	5.38	6.37	7.43	8.56
	36	0.28	0.61	1.00	1.46	2.00	2.60	3.29	4.04	4.87	5.78	6.75	7.79
6	2	1.70	3.52	5.44	7.40	9.37	11.4	13.3	15.3	17.3	19.3	21.3	23.2
	3	1.51	3.23	5.11	7.06	9.03	11.0	13.0	15.0	17.0	19.0	21.0	22.9
	4	1.35	2.96	4.79	6.70	8.67	10.7	12.7	14.6	16.6	18.6	20.6	22.6
	5	1.21	2.72	4.48	6.36	8.30	10.3	12.3	14.3	16.3	18.3	20.3	22.3
	6	1.10	2.51	4.20	6.03	7.94	9.90	11.9	13.9	15.9	17.9	19.9	21.9
	7	1.00	2.33	3.96	5.73	7.60	9.53	11.5	13.5	15.5	17.5	19.5	21.5
	8	0.92	2.18	3.73	5.45	7.27	9.17	11.1	13.1	15.1	17.1	19.1	21.1
	9	0.85	2.04	3.53	5.19	6.96	8.83	10.8	12.7	14.7	16.7	18.7	20.7
	10	0.79	1.92	3.35	4.94	6.67	8.50	10.4	12.4	14.3	16.3	18.3	20.3
	12	0.69	1.71	3.02	4.50	6.13	7.88	9.73	11.6	13.6	15.5	17.5	19.5
	14	0.61	1.55	2.75	4.12	5.65	7.33	9.11	11.0	12.9	14.8	16.8	18.8
	16	0.55	1.41	2.51	3.78	5.22	6.83	8.55	10.3	12.2	14.1	16.0	18.0
	18	0.50	1.29	2.31	3.49	4.85	6.39	8.04	9.77	11.6	13.4	15.3	17.3
	20	0.46	1.19	2.13	3.24	4.53	6.00	7.57	9.25	11.0	12.8	14.7	16.6
	24	0.40	1.03	1.84	2.82	3.99	5.32	6.76	8.32	9.97	11.7	13.5	15.3
	28	0.35	0.90	1.62	2.50	3.56	4.76	6.09	7.53	9.08	10.7	12.4	14.2
	32	0.31	0.80	1.44	2.24	3.20	4.30	5.52	6.86	8.32	9.85	11.5	13.1
	36	0.28	0.72	1.30	2.02	2.90	3.92	5.04	6.30	7.66	9.10	10.6	12.2

Table 7–10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

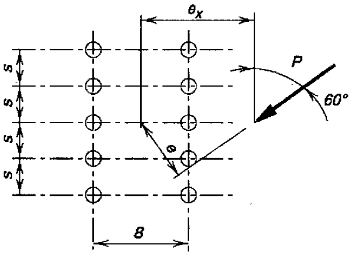
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.86	3.71	5.56	7.41	9.28	11.2	13.1	15.0	16.9	18.8	20.8	22.7
	3	1.77	3.52	5.29	7.07	8.88	10.7	12.6	14.5	16.4	18.3	20.2	22.1
	4	1.66	3.31	4.99	6.70	8.45	10.3	12.1	13.9	15.8	17.7	19.6	21.6
	5	1.54	3.10	4.70	6.34	8.04	9.79	11.6	13.4	15.3	17.1	19.0	21.0
	6	1.43	2.90	4.41	6.00	7.64	9.35	11.1	12.9	14.7	16.6	18.5	20.4
	7	1.33	2.71	4.15	5.68	7.27	8.94	10.7	12.4	14.2	16.1	17.9	19.8
	8	1.24	2.54	3.92	5.39	6.94	8.56	10.3	12.0	13.8	15.6	17.4	19.3
	9	1.16	2.38	3.70	5.12	6.63	8.22	9.86	11.6	13.3	15.1	16.9	18.7
	10	1.08	2.24	3.51	4.88	6.34	7.89	9.49	11.2	12.9	14.6	16.4	18.2
	12	0.96	2.00	3.17	4.44	5.82	7.28	8.81	10.4	12.1	13.8	15.5	17.3
	14	0.86	1.81	2.88	4.07	5.36	6.73	8.19	9.72	11.3	13.0	14.7	16.4
	16	0.77	1.64	2.64	3.74	4.95	6.25	7.64	9.11	10.7	12.2	13.9	15.6
	18	0.70	1.51	2.43	3.46	4.59	5.83	7.15	8.56	10.0	11.6	13.2	14.8
	20	0.65	1.39	2.25	3.21	4.28	5.45	6.71	8.06	9.48	11.0	12.5	14.1
	24	0.56	1.20	1.95	2.80	3.76	4.81	5.96	7.19	8.50	9.88	11.3	12.8
	28	0.49	1.06	1.72	2.48	3.34	4.29	5.34	6.47	7.68	8.97	10.3	11.7
	32	0.43	0.94	1.54	2.22	3.00	3.87	4.83	5.87	6.99	8.19	9.46	10.8
	36	0.39	0.85	1.39	2.01	2.72	3.52	4.40	5.36	6.41	7.53	8.71	9.96
6	2	1.86	3.72	5.59	7.50	9.43	11.4	13.3	15.3	17.3	19.2	21.2	23.2
	3	1.77	3.55	5.37	7.25	9.16	11.1	13.0	15.0	17.0	18.9	20.9	22.9
	4	1.66	3.36	5.14	6.98	8.88	10.8	12.7	14.7	16.7	18.6	20.6	22.6
	5	1.54	3.17	4.90	6.72	8.59	10.5	12.4	14.4	16.3	18.3	20.3	22.2
	6	1.43	2.99	4.67	6.46	8.31	10.2	12.1	14.1	16.0	18.0	19.9	21.9
	7	1.33	2.82	4.46	6.21	8.05	9.92	11.8	13.8	15.7	17.7	19.6	21.6
	8	1.24	2.67	4.26	5.98	7.79	9.65	11.5	13.5	15.4	17.3	19.3	21.3
	9	1.16	2.52	4.08	5.76	7.55	9.39	11.3	13.2	15.1	17.0	19.0	20.9
	10	1.08	2.40	3.91	5.56	7.32	9.14	11.0	12.9	14.8	16.7	18.7	20.6
	12	0.96	2.17	3.61	5.20	6.90	8.66	10.5	12.4	14.2	16.1	18.1	20.0
	14	0.86	1.98	3.35	4.87	6.51	8.23	10.0	11.8	13.7	15.6	17.5	19.4
	16	0.77	1.82	3.11	4.57	6.15	7.81	9.56	11.4	13.2	15.1	16.9	18.9
	18	0.70	1.69	2.91	4.30	5.81	7.43	9.13	10.9	12.7	14.5	16.4	18.3
	20	0.65	1.57	2.72	4.05	5.50	7.07	8.73	10.5	12.2	14.1	15.9	17.8
	24	0.56	1.37	2.41	3.61	4.96	6.43	8.00	9.67	11.4	13.2	15.0	16.8
	28	0.49	1.22	2.15	3.25	4.49	5.88	7.38	8.97	10.6	12.3	14.1	15.9
	32	0.43	1.09	1.94	2.94	4.10	5.41	6.83	8.34	9.92	11.6	13.3	15.0
	36	0.39	0.99	1.76	2.69	3.77	5.00	6.35	7.78	9.30	10.9	12.5	14.2

Table 7–10 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

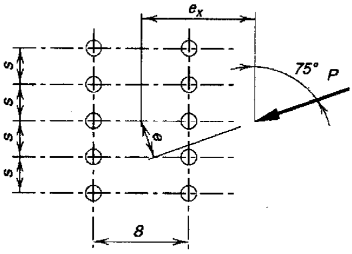
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	1	2	3	4	5	6	7	8	9	10	11	12
		Number of Bolts in One Vertical Row, *n*											
3	2	1.94	3.87	5.79	7.70	9.61	11.5	13.4	15.3	17.3	19.2	21.1	23.0
	3	1.92	3.82	5.70	7.58	9.45	11.3	13.2	15.1	17.0	18.9	20.8	22.7
	4	1.89	3.75	5.60	7.43	9.26	11.1	12.9	14.8	16.7	18.5	20.4	22.3
	5	1.85	3.67	5.48	7.28	9.07	10.9	12.7	14.5	16.4	18.2	20.1	22.0
	6	1.81	3.59	5.35	7.11	8.87	10.6	12.4	14.2	16.1	17.9	19.8	21.6
	7	1.76	3.50	5.22	6.94	8.67	10.4	12.2	14.0	15.8	17.6	19.4	21.3
	8	1.71	3.40	5.08	6.76	8.46	10.2	11.9	13.7	15.5	17.3	19.1	21.0
	9	1.66	3.30	4.94	6.59	8.26	9.96	11.7	13.4	15.2	17.0	18.8	20.6
	10	1.61	3.20	4.80	6.42	8.06	9.73	11.4	13.2	14.9	16.7	18.5	20.3
	12	1.51	3.01	4.53	6.08	7.67	9.30	11.0	12.7	14.4	16.2	17.9	19.7
	14	1.41	2.82	4.27	5.76	7.31	8.90	10.5	12.2	13.9	15.6	17.4	19.2
	16	1.31	2.65	4.03	5.47	6.96	8.52	10.1	11.8	13.4	15.2	16.9	18.6
	18	1.23	2.48	3.80	5.19	6.64	8.16	9.73	11.3	13.0	14.7	16.4	18.1
	20	1.15	2.34	3.60	4.93	6.34	7.82	9.36	10.9	12.6	14.2	15.9	17.7
	24	1.01	2.08	3.23	4.48	5.80	7.20	8.67	10.2	11.8	13.4	15.0	16.7
	28	0.90	1.87	2.93	4.08	5.33	6.65	8.06	9.52	11.0	12.6	14.2	15.9
	32	0.81	1.69	2.67	3.75	4.91	6.17	7.51	8.91	10.4	11.9	13.5	15.1
	36	0.73	1.54	2.45	3.45	4.55	5.74	7.01	8.36	9.77	11.2	12.8	14.3
6	2	1.94	3.86	5.77	7.68	9.60	11.5	13.5	15.4	17.6	19.6	21.5	23.5
	3	1.92	3.80	5.68	7.55	9.45	11.4	13.3	15.2	17.2	19.1	21.1	23.0
	4	1.89	3.74	5.57	7.42	9.29	11.2	13.1	15.0	16.9	18.9	20.8	22.8
	5	1.85	3.66	5.46	7.29	9.14	11.0	12.9	14.8	16.7	18.7	20.6	22.6
	6	1.81	3.58	5.35	7.15	8.98	10.8	12.7	14.6	16.5	18.5	20.4	22.3
	7	1.76	3.49	5.23	7.01	8.83	10.7	12.5	14.4	16.3	18.3	20.2	22.1
	8	1.71	3.40	5.12	6.88	8.68	10.5	12.4	14.3	16.2	18.1	20.0	21.9
	9	1.66	3.31	5.00	6.74	8.53	10.4	12.2	14.1	16.0	17.9	19.8	21.7
	10	1.61	3.22	4.89	6.61	8.38	10.2	12.0	13.9	15.8	17.7	19.6	21.5
	12	1.51	3.05	4.67	6.36	8.10	9.89	11.7	13.6	15.4	17.3	19.2	21.1
	14	1.41	2.88	4.46	6.12	7.84	9.61	11.4	13.3	15.1	17.0	18.9	20.8
	16	1.31	2.73	4.26	5.89	7.59	9.33	11.1	12.9	14.8	16.6	18.5	20.4
	18	1.23	2.58	4.08	5.68	7.35	9.08	10.8	12.7	14.5	16.3	18.2	20.1
	20	1.15	2.45	3.90	5.47	7.13	8.84	10.6	12.4	14.2	16.0	17.9	19.7
	24	1.01	2.21	3.59	5.10	6.71	8.38	10.1	11.9	13.6	15.5	17.3	19.1
	28	0.90	2.01	3.32	4.77	6.32	7.96	9.65	11.4	13.1	14.9	16.7	18.5
	32	0.81	1.84	3.08	4.47	5.97	7.56	9.21	10.9	12.7	14.4	16.2	18.0
	36	0.73	1.70	2.87	4.19	5.64	7.19	8.80	10.5	12.2	13.9	15.7	17.5

Table 7-11
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

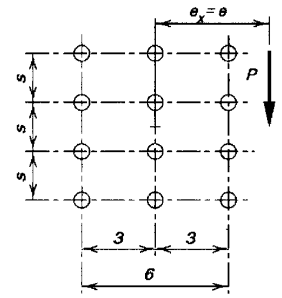
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n* 1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.71	4.07	6.81	9.86	13.0	16.1	19.3	22.3	25.4	28.5	31.5	34.5
	3	1.42	3.40	5.79	8.61	11.7	14.8	18.0	21.1	24.3	27.4	30.5	33.6
	4	1.21	2.90	4.97	7.53	10.4	13.4	16.6	19.8	23.0	26.1	29.3	32.5
	5	1.05	2.51	4.34	6.64	9.24	12.1	15.2	18.3	21.5	24.7	27.9	31.1
	6	0.92	2.21	3.85	5.91	8.27	11.0	13.9	16.9	20.0	23.2	26.4	29.7
	7	0.81	1.96	3.44	5.31	7.46	9.95	12.7	15.6	18.6	21.8	25.0	28.2
	8	0.72	1.76	3.11	4.80	6.78	9.09	11.6	14.4	17.3	20.4	23.5	26.7
	9	0.64	1.60	2.83	4.38	6.20	8.34	10.7	13.3	16.1	19.1	22.1	25.2
	10	0.58	1.46	2.59	4.02	5.71	7.70	9.91	12.4	15.0	17.9	20.8	23.8
	12	0.49	1.24	2.21	3.44	4.91	6.65	8.59	10.8	13.2	15.7	18.5	21.3
	14	0.42	1.08	1.92	3.00	4.30	5.83	7.57	9.53	11.7	14.0	16.5	19.2
	16	0.37	0.95	1.70	2.66	3.82	5.19	6.75	8.51	10.5	12.6	14.9	17.3
	18	0.33	0.85	1.52	2.39	3.43	4.67	6.08	7.68	9.45	11.4	13.5	15.8
	20	0.29	0.77	1.37	2.16	3.11	4.24	5.53	6.99	8.61	10.4	12.3	14.4
	24	0.24	0.64	1.15	1.82	2.62	3.57	4.67	5.92	7.30	8.84	10.5	12.3
	28	0.21	0.55	0.99	1.57	2.26	3.08	4.04	5.12	6.33	7.67	9.13	10.7
	32	0.18	0.49	0.87	1.38	1.98	2.71	3.55	4.51	5.58	6.77	8.06	9.47
	36	0.16	0.43	0.77	1.23	1.77	2.42	3.17	4.03	4.99	6.05	7.21	8.48
	C′	5.89	15.8	28.0	44.7	64.3	88.5	116	148	183	223	267	315
6	2	1.71	4.85	8.04	11.2	14.2	17.3	20.3	23.2	26.2	29.2	32.2	35.1
	3	1.42	4.24	7.36	10.6	13.7	16.8	19.9	22.9	25.9	28.9	31.9	34.9
	4	1.21	3.72	6.66	9.86	13.1	16.2	19.4	22.4	25.5	28.5	31.6	34.6
	5	1.05	3.29	6.00	9.14	12.4	15.6	18.7	21.9	25.0	28.1	31.1	34.2
	6	0.92	2.93	5.41	8.44	11.6	14.9	18.1	21.2	24.4	27.5	30.6	33.7
	7	0.81	2.63	4.90	7.79	10.9	14.1	17.3	20.6	23.7	26.9	30.0	33.2
	8	0.72	2.38	4.46	7.20	10.2	13.4	16.6	19.8	23.0	26.2	29.4	32.6
	9	0.64	2.17	4.09	6.67	9.54	12.6	15.8	19.1	22.3	25.5	28.7	31.9
	10	0.58	2.00	3.78	6.20	8.94	12.0	15.1	18.3	21.6	24.8	28.0	31.2
	12	0.49	1.71	3.27	5.41	7.88	10.7	13.7	16.8	20.0	23.3	26.5	29.8
	14	0.42	1.49	2.87	4.78	7.01	9.61	12.4	15.4	18.6	21.8	25.0	28.2
	16	0.37	1.32	2.55	4.28	6.29	8.69	11.3	14.2	17.2	20.3	23.5	26.7
	18	0.33	1.19	2.30	3.86	5.70	7.91	10.4	13.1	15.9	18.9	22.0	25.2
	20	0.29	1.08	2.09	3.51	5.20	7.25	9.54	12.1	14.8	17.7	20.7	23.8
	24	0.24	0.91	1.76	2.97	4.42	6.19	8.19	10.4	12.9	15.5	18.3	21.2
	28	0.21	0.78	1.52	2.57	3.84	5.39	7.14	9.15	11.4	13.7	16.3	19.0
	32	0.18	0.69	1.33	2.27	3.39	4.77	6.33	8.13	10.1	12.3	14.6	17.1
	36	0.16	0.61	1.19	2.03	3.03	4.27	5.67	7.30	9.10	11.1	13.2	15.5
	C′	5.89	22.4	43.3	74.4	112	158	212	275	345	424	510	606

Table 7-11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

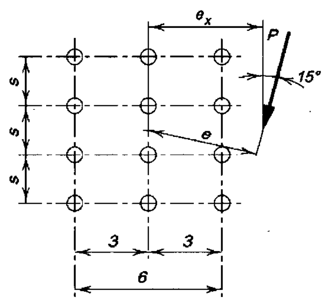
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.77	4.10	6.84	9.82	12.9	16.0	19.1	22.2	25.2	28.3	31.3	34.3
	3	1.47	3.45	5.86	8.61	11.6	14.7	17.8	20.9	24.1	27.2	30.3	33.3
	4	1.25	2.95	5.07	7.55	10.4	13.3	16.4	19.5	22.7	25.8	29.0	32.1
	5	1.08	2.57	4.44	6.67	9.26	12.1	15.1	18.1	21.3	24.4	27.6	30.7
	6	0.94	2.26	3.93	5.96	8.33	11.0	13.8	16.8	19.8	23.0	26.1	29.3
	7	0.83	2.01	3.52	5.37	7.55	9.97	12.7	15.5	18.5	21.5	24.7	27.8
	8	0.74	1.81	3.18	4.87	6.88	9.13	11.7	14.4	17.2	20.2	23.2	26.4
	9	0.66	1.64	2.90	4.45	6.31	8.40	10.8	13.3	16.1	18.9	21.9	25.0
	10	0.60	1.50	2.65	4.10	5.81	7.77	9.99	12.4	15.0	17.8	20.7	23.6
	12	0.50	1.28	2.27	3.52	5.01	6.74	8.71	10.9	13.2	15.8	18.4	21.2
	14	0.43	1.11	1.98	3.08	4.40	5.93	7.69	9.62	11.8	14.1	16.5	19.1
	16	0.38	0.98	1.75	2.73	3.91	5.29	6.87	8.62	10.6	12.7	15.0	17.4
	18	0.34	0.88	1.57	2.45	3.52	4.77	6.20	7.80	9.59	11.5	13.6	15.9
	20	0.30	0.79	1.42	2.22	3.19	4.33	5.65	7.12	8.76	10.5	12.5	14.6
	24	0.25	0.67	1.19	1.87	2.69	3.66	4.78	6.04	7.45	8.99	10.7	12.5
	28	0.22	0.57	1.02	1.61	2.32	3.17	4.14	5.24	6.47	7.82	9.31	10.9
	32	0.19	0.50	0.90	1.42	2.04	2.79	3.65	4.62	5.72	6.92	8.24	9.66
	36	0.17	0.45	0.80	1.26	1.82	2.49	3.26	4.13	5.11	6.20	7.38	8.66
6	2	1.77	4.83	7.98	11.1	14.1	17.2	20.2	23.2	26.1	29.1	32.1	35.0
	3	1.47	4.22	7.31	10.5	13.6	16.7	19.7	22.8	25.8	28.8	31.8	34.8
	4	1.25	3.71	6.64	9.77	12.9	16.1	19.2	22.3	25.3	28.3	31.4	34.4
	5	1.08	3.28	6.01	9.06	12.2	15.4	18.5	21.7	24.8	27.8	30.9	33.9
	6	0.94	2.94	5.45	8.38	11.5	14.7	17.8	21.0	24.1	27.2	30.3	33.4
	7	0.83	2.65	4.97	7.75	10.8	13.9	17.1	20.3	23.5	26.6	29.7	32.8
	8	0.74	2.40	4.55	7.17	10.1	13.2	16.4	19.6	22.7	25.9	29.1	32.2
	9	0.66	2.20	4.18	6.66	9.49	12.5	15.6	18.8	22.0	25.2	28.4	31.5
	10	0.60	2.02	3.86	6.20	8.92	11.9	14.9	18.1	21.3	24.5	27.6	30.8
	12	0.50	1.74	3.34	5.43	7.91	10.6	13.6	16.6	19.8	23.0	26.1	29.3
	14	0.43	1.52	2.94	4.82	7.07	9.60	12.4	15.3	18.4	21.5	24.6	27.8
	16	0.38	1.35	2.62	4.32	6.38	8.71	11.3	14.1	17.0	20.1	23.2	26.3
	18	0.34	1.22	2.36	3.91	5.79	7.95	10.4	13.0	15.8	18.8	21.8	24.9
	20	0.30	1.10	2.14	3.57	5.30	7.31	9.60	12.1	14.8	17.6	20.5	23.5
	24	0.25	0.93	1.81	3.03	4.52	6.26	8.28	10.5	12.9	15.5	18.2	21.1
	28	0.22	0.80	1.56	2.63	3.93	5.47	7.26	9.24	11.4	13.8	16.3	18.9
	32	0.19	0.71	1.37	2.32	3.47	4.85	6.45	8.23	10.2	12.4	14.7	17.1
	36	0.17	0.63	1.23	2.08	3.11	4.35	5.80	7.41	9.23	11.2	13.3	15.6

Table 7–11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

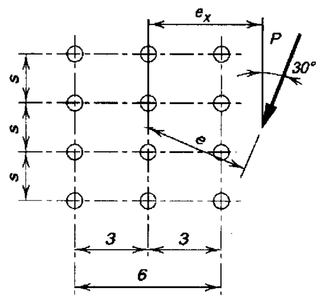
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	1.94	4.26	6.99	9.90	12.9	16.0	19.0	22.0	25.1	28.1	31.1	34.1
	3	1.61	3.63	6.09	8.80	11.7	14.7	17.7	20.8	23.9	27.0	30.0	33.1
	4	1.37	3.15	5.35	7.83	10.6	13.5	16.5	19.5	22.6	25.7	28.7	31.8
	5	1.19	2.77	4.74	7.00	9.54	12.3	15.2	18.2	21.2	24.3	27.4	30.5
	6	1.04	2.45	4.23	6.30	8.67	11.3	14.1	17.0	19.9	23.0	26.0	29.1
	7	0.92	2.19	3.81	5.71	7.92	10.4	13.0	15.8	18.7	21.7	24.7	27.8
	8	0.82	1.98	3.45	5.22	7.27	9.58	12.1	14.8	17.6	20.5	23.4	26.4
	9	0.74	1.80	3.16	4.79	6.71	8.88	11.2	13.8	16.5	19.3	22.2	25.2
	10	0.67	1.65	2.90	4.42	6.22	8.26	10.5	12.9	15.5	18.2	21.1	24.0
	12	0.56	1.41	2.49	3.82	5.41	7.22	9.23	11.5	13.8	16.4	19.0	21.8
	14	0.48	1.23	2.18	3.36	4.78	6.40	8.22	10.3	12.4	14.8	17.2	19.8
	16	0.42	1.08	1.93	2.99	4.26	5.73	7.40	9.25	11.3	13.4	15.7	18.2
	18	0.38	0.97	1.73	2.69	3.85	5.18	6.71	8.41	10.3	12.3	14.4	16.7
	20	0.34	0.88	1.57	2.44	3.50	4.73	6.14	7.70	9.42	11.3	13.3	15.4
	24	0.28	0.74	1.32	2.06	2.96	4.01	5.22	6.58	8.08	9.72	11.5	13.4
	28	0.24	0.64	1.14	1.78	2.56	3.48	4.54	5.73	7.05	8.51	10.1	11.8
	32	0.21	0.56	1.00	1.57	2.26	3.07	4.01	5.07	6.25	7.55	8.96	10.5
	36	0.19	0.50	0.89	1.40	2.02	2.75	3.59	4.54	5.61	6.78	8.06	9.44
6	2	1.94	4.86	7.96	11.0	14.1	17.1	20.1	23.1	26.0	29.0	32.0	35.0
	3	1.61	4.27	7.32	10.4	13.5	16.6	19.6	22.6	25.6	28.6	31.6	34.6
	4	1.37	3.78	6.70	9.75	12.9	15.9	19.0	22.1	25.1	28.1	31.1	34.2
	5	1.19	3.39	6.14	9.10	12.2	15.3	18.4	21.5	24.5	27.6	30.6	33.7
	6	1.04	3.06	5.64	8.48	11.5	14.6	17.7	20.8	23.9	27.0	30.1	33.1
	7	0.92	2.78	5.19	7.91	10.9	13.9	17.0	20.1	23.2	26.3	29.4	32.5
	8	0.82	2.54	4.80	7.38	10.3	13.3	16.3	19.4	22.6	25.7	28.8	31.9
	9	0.74	2.34	4.45	6.90	9.67	12.6	15.7	18.7	21.9	25.0	28.1	31.2
	10	0.67	2.16	4.14	6.46	9.14	12.0	15.0	18.1	21.2	24.3	27.4	30.5
	12	0.56	1.87	3.61	5.71	8.20	10.9	13.8	16.8	19.8	22.9	26.0	29.1
	14	0.48	1.65	3.20	5.10	7.41	9.95	12.7	15.6	18.5	21.5	24.6	27.7
	16	0.42	1.47	2.86	4.60	6.74	9.12	11.7	14.5	17.3	20.3	23.3	26.4
	18	0.38	1.33	2.58	4.19	6.17	8.39	10.8	13.5	16.2	19.1	22.0	25.0
	20	0.34	1.21	2.35	3.84	5.68	7.75	10.1	12.6	15.2	18.0	20.9	23.8
	24	0.28	1.02	2.00	3.29	4.89	6.71	8.78	11.1	13.5	16.1	18.8	21.6
	28	0.24	0.88	1.73	2.86	4.28	5.90	7.77	9.83	12.1	14.5	17.0	19.6
	32	0.21	0.78	1.52	2.54	3.80	5.25	6.95	8.83	10.9	13.1	15.4	17.9
	36	0.19	0.70	1.36	2.27	3.41	4.73	6.28	8.00	9.88	11.9	14.1	16.4

Table 7–11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

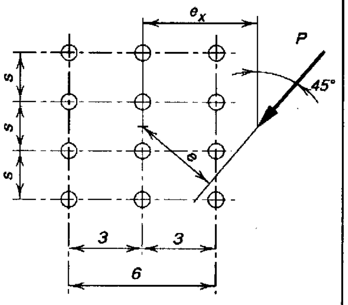
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.23	4.67	7.33	10.2	13.1	16.0	19.0	22.0	25.0	28.0	31.0	33.9
	3	1.89	4.06	6.50	9.19	12.0	14.9	17.9	20.9	23.9	26.9	29.9	32.9
	4	1.63	3.57	5.84	8.36	11.1	13.9	16.8	19.7	22.7	25.7	28.7	31.7
	5	1.42	3.17	5.27	7.63	10.2	12.9	15.7	18.6	21.5	24.5	27.5	30.5
	6	1.25	2.84	4.78	6.99	9.40	12.0	14.7	17.6	20.4	23.4	26.3	29.3
	7	1.11	2.57	4.36	6.42	8.70	11.2	13.8	16.6	19.4	22.3	25.2	28.2
	8	0.99	2.33	3.99	5.92	8.09	10.5	13.0	15.7	18.4	21.2	24.1	27.0
	9	0.90	2.13	3.68	5.49	7.54	9.80	12.2	14.8	17.5	20.3	23.1	26.0
	10	0.81	1.96	3.40	5.10	7.05	9.21	11.6	14.0	16.6	19.3	22.1	24.9
	12	0.68	1.68	2.95	4.46	6.22	8.19	10.4	12.7	15.1	17.7	20.3	23.0
	14	0.59	1.47	2.59	3.95	5.55	7.35	9.34	11.5	13.8	16.2	18.7	21.3
	16	0.52	1.31	2.31	3.54	4.99	6.65	8.49	10.5	12.7	14.9	17.3	19.8
	18	0.46	1.17	2.08	3.20	4.54	6.06	7.77	9.64	11.7	13.8	16.1	18.5
	20	0.41	1.06	1.89	2.92	4.15	5.56	7.15	8.90	10.8	12.8	15.0	17.2
	24	0.35	0.90	1.60	2.48	3.54	4.76	6.15	7.70	9.39	11.2	13.1	15.2
	28	0.30	0.77	1.38	2.15	3.08	4.16	5.39	6.77	8.28	9.91	11.7	13.5
	32	0.26	0.68	1.22	1.90	2.72	3.68	4.79	6.03	7.39	8.87	10.5	12.2
	36	0.23	0.61	1.08	1.69	2.44	3.30	4.30	5.42	6.66	8.02	9.49	11.1
6	2	2.23	5.02	8.01	11.0	14.0	17.0	20.0	23.0	25.9	28.9	31.9	34.8
	3	1.89	4.50	7.44	10.4	13.5	16.5	19.5	22.5	25.5	28.4	31.4	34.4
	4	1.63	4.05	6.89	9.86	12.9	15.9	18.9	21.9	24.9	27.9	30.9	33.9
	5	1.42	3.68	6.40	9.30	12.3	15.3	18.3	21.3	24.4	27.4	30.4	33.4
	6	1.25	3.36	5.96	8.78	11.7	14.7	17.7	20.7	23.8	26.8	29.8	32.8
	7	1.11	3.09	5.57	8.29	11.2	14.1	17.1	20.1	23.2	26.2	29.2	32.3
	8	0.99	2.86	5.22	7.84	10.6	13.6	16.5	19.5	22.6	25.6	28.6	31.7
	9	0.90	2.65	4.90	7.43	10.2	13.0	16.0	19.0	22.0	25.0	28.0	31.1
	10	0.81	2.47	4.61	7.04	9.69	12.5	15.4	18.4	21.4	24.4	27.4	30.4
	12	0.68	2.16	4.11	6.35	8.85	11.6	14.4	17.3	20.2	23.2	26.2	29.2
	14	0.59	1.92	3.69	5.76	8.11	10.7	13.4	16.2	19.1	22.1	25.0	28.0
	16	0.52	1.72	3.34	5.25	7.47	9.94	12.6	15.3	18.1	21.0	23.9	26.9
	18	0.46	1.56	3.04	4.82	6.91	9.26	11.8	14.4	17.2	20.0	22.9	25.8
	20	0.41	1.43	2.79	4.44	6.43	8.66	11.1	13.6	16.3	19.0	21.9	24.7
	24	0.35	1.22	2.38	3.84	5.62	7.64	9.84	12.2	14.7	17.3	20.0	22.8
	28	0.30	1.06	2.08	3.37	4.98	6.81	8.82	11.0	13.4	15.8	18.4	21.1
	32	0.26	0.94	1.84	3.00	4.46	6.12	7.97	10.0	12.2	14.6	17.0	19.5
	36	0.23	0.84	1.65	2.71	4.04	5.56	7.27	9.18	11.2	13.4	15.7	18.1

Table 7-11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

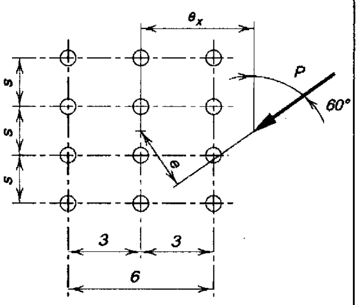
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.59	5.21	7.88	10.6	13.4	16.3	19.2	22.1	25.0	28.0	30.9	33.9
	3	2.32	4.73	7.27	9.91	12.7	15.5	18.3	21.2	24.1	27.0	30.0	32.9
	4	2.07	4.29	6.69	9.23	11.9	14.6	17.5	20.3	23.2	26.1	29.0	32.0
	5	1.84	3.90	6.18	8.63	11.2	13.9	16.6	19.5	22.3	25.2	28.1	31.0
	6	1.65	3.56	5.73	8.08	10.6	13.2	15.9	18.7	21.5	24.3	27.2	30.1
	7	1.49	3.27	5.32	7.59	10.0	12.6	15.2	17.9	20.7	23.5	26.3	29.2
	8	1.35	3.01	4.95	7.13	9.48	12.0	14.5	17.2	19.9	22.7	25.5	28.4
	9	1.23	2.78	4.63	6.71	8.98	11.4	13.9	16.5	19.2	22.0	24.7	27.6
	10	1.12	2.58	4.34	6.33	8.52	10.9	13.3	15.9	18.5	21.2	24.0	26.8
	12	0.95	2.25	3.84	5.67	7.70	9.91	12.3	14.7	17.3	19.9	22.6	25.3
	14	0.83	1.98	3.43	5.11	7.00	9.08	11.3	13.7	16.1	18.7	21.3	23.9
	16	0.73	1.77	3.09	4.64	6.40	8.36	10.5	12.7	15.1	17.5	20.1	22.6
	18	0.65	1.60	2.81	4.24	5.89	7.73	9.74	11.9	14.2	16.5	19.0	21.5
	20	0.59	1.46	2.57	3.9	5.44	7.19	9.09	11.1	13.3	15.6	17.9	20.4
	24	0.49	1.24	2.20	3.35	4.72	6.27	7.99	9.85	11.9	14.0	16.2	18.5
	28	0.42	1.07	1.91	2.93	4.15	5.55	7.10	8.81	10.7	12.6	14.7	16.8
	32	0.37	0.95	1.69	2.60	3.70	4.97	6.38	7.95	9.65	11.5	13.4	15.4
	36	0.33	0.85	1.51	2.34	3.34	4.49	5.79	7.23	8.81	10.5	12.3	14.2
6	2	2.59	5.32	8.17	11.1	14.0	17.0	19.9	22.9	25.8	28.8	31.8	34.7
	3	2.32	4.94	7.73	10.6	13.5	16.5	19.4	22.4	25.4	28.3	31.3	34.3
	4	2.07	4.57	7.31	10.2	13.1	16.0	19.0	21.9	24.9	27.8	30.8	33.8
	5	1.84	4.25	6.91	9.73	12.6	15.5	18.5	21.4	24.4	27.4	30.3	33.3
	6	1.65	3.95	6.55	9.32	12.2	15.1	18.0	20.9	23.9	26.9	29.8	32.8
	7	1.49	3.69	6.22	8.94	11.8	14.6	17.5	20.5	23.4	26.4	29.3	32.3
	8	1.35	3.46	5.92	8.58	11.4	14.2	17.1	20.0	22.9	25.9	28.8	31.8
	9	1.23	3.25	5.64	8.25	11.0	13.8	16.7	19.6	22.5	25.4	28.4	31.3
	10	1.12	3.06	5.39	7.94	10.6	13.4	16.3	19.1	22.0	24.9	27.9	30.8
	12	0.95	2.73	4.92	7.37	9.97	12.7	15.5	18.3	21.2	24.1	27.0	29.9
	14	0.83	2.46	4.52	6.85	9.36	12.0	14.7	17.5	20.3	23.2	26.1	29.0
	16	0.73	2.23	4.18	6.39	8.80	11.4	14.0	16.8	19.6	22.4	25.3	28.1
	18	0.65	2.04	3.87	5.97	8.28	10.8	13.4	16.1	18.8	21.6	24.4	27.3
	20	0.59	1.88	3.60	5.59	7.81	10.2	12.8	15.4	18.1	20.9	23.7	26.5
	24	0.49	1.63	3.15	4.94	6.99	9.25	11.7	14.2	16.8	19.5	22.2	25.0
	28	0.42	1.43	2.79	4.41	6.31	8.44	10.7	13.1	15.7	18.2	20.9	23.6
	32	0.37	1.27	2.49	3.97	5.74	7.74	9.90	12.2	14.6	17.1	19.7	22.3
	36	0.33	1.15	2.25	3.61	5.26	7.13	9.17	11.4	13.7	16.1	18.6	21.1

Table 7–11 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

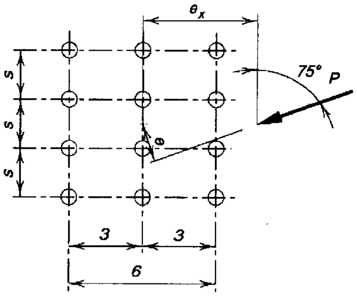
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.86	5.68	8.47	11.3	14.1	16.9	19.8	22.6	25.5	28.4	31.3	34.2
	3	2.77	5.49	8.19	10.9	13.7	16.4	19.2	22.1	24.9	27.8	30.7	33.6
	4	2.66	5.27	7.89	10.5	13.2	16.0	18.8	21.6	24.4	27.2	30.1	33.0
	5	2.53	5.04	7.58	10.2	12.8	15.5	18.3	21.0	23.9	26.7	29.5	32.4
	6	2.40	4.81	7.27	9.81	12.4	15.1	17.8	20.6	23.3	26.2	29.0	31.8
	7	2.26	4.57	6.97	9.47	12.0	14.7	17.4	20.1	22.9	25.6	28.4	31.3
	8	2.13	4.35	6.69	9.13	11.7	14.3	16.9	19.6	22.4	25.1	27.9	30.7
	9	2.00	4.13	6.41	8.82	11.3	13.9	16.5	19.2	21.9	24.7	27.4	30.2
	10	1.89	3.93	6.15	8.51	11.0	13.5	16.1	18.8	21.5	24.2	27.0	29.8
	12	1.67	3.57	5.67	7.95	10.4	12.9	15.4	18.0	20.7	23.4	26.1	28.8
	14	1.49	3.25	5.25	7.44	9.77	12.2	14.7	17.3	19.9	22.6	25.3	28.0
	16	1.34	2.97	4.87	6.98	9.23	11.6	14.1	16.6	19.2	21.8	24.5	27.2
	18	1.21	2.73	4.54	6.56	8.74	11.1	13.5	16.0	18.5	21.1	23.7	26.4
	20	1.10	2.53	4.24	6.18	8.28	10.5	12.9	15.3	17.8	20.4	23.0	25.6
	24	0.93	2.19	3.75	5.52	7.48	9.59	11.8	14.2	16.6	19.1	21.6	24.2
	28	0.80	1.93	3.34	4.97	6.79	8.78	10.9	13.2	15.5	17.9	20.4	22.9
	32	0.71	1.72	3.01	4.51	6.20	8.08	10.1	12.3	14.5	16.8	19.2	21.7
	36	0.63	1.55	2.74	4.12	5.70	7.47	9.40	11.5	13.6	15.9	18.2	20.6
6	2	2.86	5.66	8.48	11.3	14.2	17.1	20.1	23.0	26.4	29.3	32.3	35.2
	3	2.77	5.49	8.25	11.1	13.9	16.8	19.7	22.7	25.6	28.5	31.5	34.4
	4	2.66	5.30	8.02	10.8	13.6	16.5	19.4	22.3	25.2	28.2	31.1	34.0
	5	2.53	5.10	7.79	10.6	13.4	16.2	19.1	22.0	24.9	27.8	30.8	33.7
	6	2.40	4.91	7.56	10.3	13.1	15.9	18.8	21.7	24.6	27.5	30.4	33.3
	7	2.26	4.72	7.34	10.1	12.9	15.7	18.5	21.4	24.3	27.2	30.1	33.0
	8	2.13	4.54	7.14	9.83	12.6	15.4	18.3	21.1	24.0	26.9	29.8	32.7
	9	2.00	4.37	6.94	9.61	12.4	15.2	18.0	20.8	23.7	26.6	29.5	32.4
	10	1.89	4.21	6.75	9.40	12.1	14.9	17.7	20.6	23.4	26.3	29.2	32.1
	12	1.67	3.90	6.39	9.00	11.7	14.4	17.2	20.0	22.9	25.7	28.6	31.5
	14	1.49	3.63	6.06	8.63	11.3	14.0	16.8	19.6	22.4	25.2	28.1	30.9
	16	1.34	3.39	5.75	8.29	10.9	13.6	16.3	19.1	21.9	24.7	27.5	30.4
	18	1.21	3.17	5.47	7.96	10.6	13.2	15.9	18.7	21.4	24.2	27.0	29.9
	20	1.10	2.98	5.22	7.66	10.2	12.9	15.5	18.2	21.0	23.8	26.6	29.4
	24	0.93	2.65	4.76	7.10	9.57	12.2	14.8	17.5	20.2	22.9	25.7	28.5
	28	0.80	2.38	4.37	6.60	8.99	11.5	14.1	16.7	19.4	22.1	24.8	27.6
	32	0.71	2.16	4.03	6.15	8.45	10.9	13.4	16.0	18.7	21.3	24.0	26.8
	36	0.63	1.97	3.73	5.75	7.96	10.3	12.8	15.3	17.9	20.6	23.3	26.0

Table 7–12
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

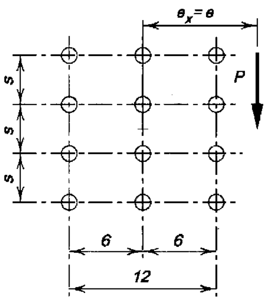
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.15	4.55	7.17	10.0	13.0	16.0	19.1	22.2	25.3	28.3	31.4	34.4
	3	1.91	4.06	6.43	9.06	11.9	14.9	17.9	21.0	24.1	27.2	30.3	33.4
	4	1.71	3.65	5.80	8.23	10.9	13.7	16.7	19.8	22.9	26.0	29.1	32.3
	5	1.55	3.31	5.27	7.51	9.97	12.7	15.5	18.5	21.5	24.7	27.8	31.0
	6	1.42	3.02	4.82	6.88	9.16	11.7	14.4	17.3	20.3	23.3	26.4	29.6
	7	1.31	2.77	4.44	6.34	8.46	10.8	13.4	16.1	19.0	22.0	25.1	28.2
	8	1.21	2.56	4.10	5.87	7.85	10.1	12.5	15.1	17.9	20.7	23.7	26.8
	9	1.12	2.38	3.81	5.46	7.31	9.39	11.7	14.1	16.8	19.6	22.5	25.5
	10	1.05	2.21	3.55	5.09	6.84	8.79	10.9	13.3	15.8	18.5	21.3	24.2
	12	0.92	1.94	3.12	4.48	6.03	7.78	9.70	11.8	14.1	16.6	19.1	21.9
	14	0.81	1.72	2.77	3.99	5.38	6.95	8.69	10.6	12.7	14.9	17.3	19.9
	16	0.72	1.53	2.48	3.58	4.84	6.27	7.85	9.60	11.5	13.6	15.8	18.1
	18	0.64	1.38	2.25	3.25	4.40	5.70	7.15	8.75	10.5	12.4	14.4	16.6
	20	0.58	1.26	2.05	2.96	4.02	5.21	6.55	8.03	9.65	11.4	13.3	15.3
	24	0.49	1.06	1.73	2.52	3.42	4.45	5.60	6.88	8.29	9.82	11.5	13.2
	28	0.42	0.92	1.50	2.19	2.97	3.87	4.88	6.00	7.24	8.59	10.1	11.6
	32	0.37	0.81	1.32	1.93	2.63	3.42	4.32	5.32	6.42	7.62	8.93	10.3
	36	0.33	0.72	1.18	1.72	2.35	3.06	3.87	4.77	5.76	6.84	8.02	9.29
	C′	11.8	26.5	43.3	63.7	86.8	114	144	178	216	257	302	352
6	2	2.15	4.94	7.98	11.1	14.2	17.2	20.2	23.2	26.2	29.2	32.1	35.1
	3	1.91	4.48	7.39	10.5	13.6	16.7	19.8	22.8	25.8	28.9	31.9	34.8
	4	1.71	4.07	6.81	9.86	13.0	16.1	19.3	22.3	25.4	28.5	31.5	34.5
	5	1.55	3.71	6.27	9.22	12.3	15.5	18.6	21.8	24.9	28.0	31.0	34.1
	6	1.42	3.40	5.79	8.61	11.7	14.8	18.0	21.1	24.3	27.4	30.5	33.6
	7	1.31	3.13	5.35	8.05	11.0	14.1	17.3	20.5	23.6	26.8	29.9	33.1
	8	1.21	2.90	4.97	7.53	10.4	13.4	16.6	19.8	23.0	26.1	29.3	32.5
	9	1.12	2.69	4.64	7.07	9.78	12.8	15.9	19.0	22.2	25.4	28.6	31.8
	10	1.05	2.51	4.34	6.64	9.24	12.1	15.2	18.3	21.5	24.7	27.9	31.1
	12	0.92	2.21	3.85	5.91	8.27	11.0	13.9	16.9	20.0	23.2	26.4	29.7
	14	0.81	1.96	3.44	5.31	7.46	9.95	12.7	15.6	18.6	21.8	25.0	28.2
	16	0.72	1.76	3.11	4.80	6.78	9.09	11.6	14.4	17.3	20.4	23.5	26.7
	18	0.64	1.60	2.83	4.38	6.20	8.34	10.7	13.3	16.1	19.1	22.1	25.2
	20	0.58	1.46	2.59	4.02	5.71	7.70	9.91	12.4	15.0	17.9	20.8	23.8
	24	0.49	1.24	2.21	3.44	4.91	6.65	8.59	10.8	13.2	15.7	18.5	21.3
	28	0.42	1.08	1.92	3.00	4.30	5.83	7.57	9.53	11.7	14.0	16.5	19.2
	32	0.37	0.95	1.70	2.66	3.82	5.19	6.75	8.51	10.5	12.6	14.9	17.3
	36	0.33	0.85	1.52	2.39	3.43	4.67	6.08	7.68	9.45	11.4	13.5	15.8
	C′	11.8	31.6	56.1	89.4	129	177	232	296	366	446	533	629

Table 7-12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

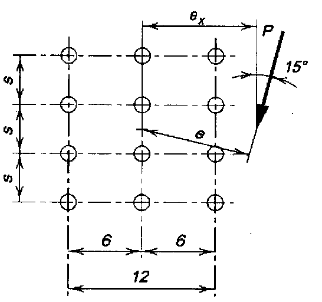
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.22	4.62	7.25	10.1	13.0	16.0	19.0	22.1	25.1	28.2	31.2	34.2
	3	1.97	4.13	6.53	9.13	11.9	14.9	17.9	20.9	24.0	27.1	30.1	33.2
	4	1.77	3.72	5.91	8.31	10.9	13.7	16.7	19.7	22.7	25.8	28.9	32.0
	5	1.61	3.38	5.39	7.60	10.1	12.7	15.5	18.4	21.4	24.5	27.6	30.7
	6	1.47	3.10	4.93	6.98	9.28	11.8	14.4	17.2	20.2	23.2	26.2	29.3
	7	1.35	2.85	4.54	6.45	8.59	10.9	13.5	16.1	19.0	21.9	24.9	27.9
	8	1.25	2.63	4.21	5.98	7.98	10.2	12.6	15.1	17.8	20.7	23.6	26.6
	9	1.16	2.44	3.91	5.57	7.45	9.51	11.8	14.2	16.8	19.5	22.4	25.3
	10	1.08	2.28	3.65	5.21	6.97	8.92	11.1	13.4	15.9	18.5	21.2	24.1
	12	0.94	2.00	3.20	4.59	6.16	7.91	9.84	11.9	14.2	16.6	19.2	21.9
	14	0.83	1.77	2.85	4.09	5.50	7.08	8.84	10.8	12.8	15.0	17.4	19.9
	16	0.74	1.58	2.56	3.68	4.96	6.40	8.00	9.75	11.7	13.7	15.9	18.2
	18	0.66	1.43	2.31	3.34	4.51	5.83	7.30	8.91	10.7	12.6	14.6	16.8
	20	0.60	1.30	2.11	3.05	4.13	5.34	6.70	8.19	9.82	11.6	13.5	15.5
	24	0.50	1.10	1.79	2.59	3.52	4.56	5.74	7.03	8.45	10.0	11.7	13.4
	28	0.43	0.95	1.55	2.25	3.06	3.98	5.01	6.15	7.40	8.77	10.2	11.8
	32	0.38	0.84	1.37	1.99	2.70	3.52	4.43	5.45	6.57	7.79	9.12	10.5
	36	0.34	0.75	1.22	1.78	2.42	3.15	3.98	4.89	5.90	7.01	8.20	9.49
6	2	2.22	4.97	7.97	11.0	14.1	17.1	20.1	23.1	26.1	29.1	32.1	35.0
	3	1.97	4.50	7.40	10.5	13.5	16.6	19.7	22.7	25.7	28.7	31.7	34.7
	4	1.77	4.10	6.84	9.82	12.9	16.0	19.1	22.2	25.2	28.3	31.3	34.3
	5	1.61	3.75	6.32	9.20	12.3	15.4	18.5	21.6	24.7	27.8	30.8	33.9
	6	1.47	3.45	5.86	8.61	11.6	14.7	17.8	20.9	24.1	27.2	30.3	33.3
	7	1.35	3.18	5.44	8.06	11.0	14.0	17.1	20.3	23.4	26.5	29.6	32.7
	8	1.25	2.95	5.07	7.55	10.4	13.3	16.4	19.5	22.7	25.8	29.0	32.1
	9	1.16	2.75	4.73	7.09	9.78	12.7	15.7	18.8	22.0	25.1	28.3	31.4
	10	1.08	2.57	4.44	6.67	9.26	12.1	15.1	18.1	21.3	24.4	27.6	30.7
	12	0.94	2.26	3.93	5.96	8.33	11.0	13.8	16.8	19.8	23.0	26.1	29.3
	14	0.83	2.01	3.52	5.37	7.55	9.97	12.7	15.5	18.5	21.5	24.7	27.8
	16	0.74	1.81	3.18	4.87	6.88	9.13	11.7	14.4	17.2	20.2	23.2	26.4
	18	0.66	1.64	2.90	4.45	6.31	8.40	10.8	13.3	16.1	18.9	21.9	25.0
	20	0.60	1.50	2.65	4.10	5.81	7.77	9.99	12.4	15.0	17.8	20.7	23.6
	24	0.50	1.28	2.27	3.52	5.01	6.74	8.71	10.9	13.2	15.8	18.4	21.2
	28	0.43	1.11	1.98	3.08	4.40	5.93	7.69	9.62	11.8	14.1	16.5	19.1
	32	0.38	0.98	1.75	2.73	3.91	5.29	6.87	8.62	10.6	12.7	15.0	17.4
	36	0.34	0.88	1.57	2.45	3.52	4.77	6.20	7.80	9.59	11.5	13.6	15.9

Table 7–12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

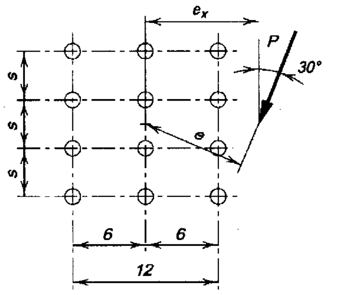
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	*e*$_x$, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.40	4.89	7.53	10.3	13.2	16.1	19.1	22.1	25.1	28.1	31.1	34.1
	3	2.15	4.40	6.84	9.45	12.2	15.1	18.0	21.0	24.0	27.0	30.0	33.0
	4	1.94	3.99	6.24	8.69	11.3	14.0	16.9	19.8	22.8	25.8	28.8	31.9
	5	1.76	3.65	5.74	8.02	10.5	13.1	15.8	18.7	21.6	24.6	27.6	30.6
	6	1.61	3.35	5.29	7.42	9.72	12.2	14.8	17.6	20.4	23.4	26.3	29.3
	7	1.49	3.10	4.90	6.89	9.06	11.4	13.9	16.6	19.3	22.2	25.1	28.1
	8	1.37	2.87	4.55	6.42	8.47	10.7	13.1	15.6	18.3	21.1	23.9	26.9
	9	1.28	2.67	4.24	6.00	7.94	10.1	12.4	14.8	17.4	20.0	22.8	25.7
	10	1.19	2.49	3.97	5.63	7.47	9.49	11.7	14.0	16.5	19.1	21.8	24.6
	12	1.04	2.19	3.50	4.98	6.64	8.48	10.5	12.6	14.9	17.3	19.9	22.5
	14	0.92	1.95	3.12	4.46	5.97	7.64	9.46	11.4	13.6	15.8	18.2	20.7
	16	0.82	1.75	2.81	4.03	5.40	6.93	8.61	10.4	12.4	14.5	16.7	19.1
	18	0.74	1.58	2.55	3.66	4.92	6.33	7.89	9.59	11.4	13.4	15.5	17.7
	20	0.67	1.44	2.33	3.35	4.52	5.82	7.27	8.85	10.6	12.4	14.4	16.4
	24	0.56	1.22	1.98	2.86	3.87	5.00	6.26	7.65	9.16	10.8	12.5	14.4
	28	0.48	1.06	1.72	2.49	3.37	4.37	5.48	6.71	8.06	9.51	11.1	12.8
	32	0.42	0.93	1.52	2.20	2.99	3.88	4.87	5.97	7.18	8.49	9.91	11.4
	36	0.38	0.83	1.36	1.97	2.68	3.48	4.38	5.38	6.47	7.66	8.95	10.3
6	2	2.40	5.11	8.05	11.1	14.1	17.1	20.1	23.0	26.0	29.0	32.0	34.9
	3	2.15	4.66	7.51	10.5	13.5	16.5	19.6	22.6	25.6	28.6	31.6	34.6
	4	1.94	4.26	6.99	9.90	12.9	16.0	19.0	22.0	25.1	28.1	31.1	34.1
	5	1.76	3.92	6.52	9.34	12.3	15.3	18.4	21.5	24.5	27.6	30.6	33.6
	6	1.61	3.63	6.09	8.80	11.7	14.7	17.7	20.8	23.9	27.0	30.0	33.1
	7	1.49	3.38	5.70	8.30	11.1	14.1	17.1	20.2	23.2	26.3	29.4	32.5
	8	1.37	3.15	5.35	7.83	10.6	13.5	16.5	19.5	22.6	25.7	28.7	31.8
	9	1.28	2.95	5.03	7.40	10.0	12.9	15.8	18.8	21.9	25.0	28.1	31.2
	10	1.19	2.77	4.74	7.00	9.54	12.3	15.2	18.2	21.2	24.3	27.4	30.5
	12	1.04	2.45	4.23	6.30	8.67	11.3	14.1	17.0	19.9	23.0	26.0	29.1
	14	0.92	2.19	3.81	5.71	7.92	10.4	13.0	15.8	18.7	21.7	24.7	27.8
	16	0.82	1.98	3.45	5.22	7.27	9.58	12.1	14.8	17.6	20.5	23.4	26.4
	18	0.74	1.80	3.16	4.79	6.71	8.88	11.2	13.8	16.5	19.3	22.2	25.2
	20	0.67	1.65	2.90	4.42	6.22	8.26	10.5	12.9	15.5	18.2	21.1	24.0
	24	0.56	1.41	2.49	3.82	5.41	7.22	9.23	11.5	13.8	16.4	19.0	21.8
	28	0.48	1.23	2.18	3.36	4.78	6.40	8.22	10.3	12.4	14.8	17.2	19.8
	32	0.42	1.08	1.93	2.99	4.26	5.73	7.40	9.25	11.3	13.4	15.7	18.2
	36	0.38	0.97	1.73	2.69	3.85	5.18	6.71	8.41	10.3	12.3	14.4	16.7

Table 7-12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

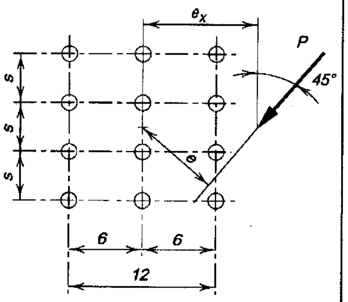
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.64	5.30	8.01	10.8	13.6	16.4	19.3	22.3	25.2	28.1	31.1	34.0
	3	2.43	4.90	7.44	10.1	12.8	15.6	18.4	21.3	24.2	27.1	30.1	33.1
	4	2.23	4.52	6.89	9.38	12.0	14.7	17.5	20.3	23.2	26.1	29.0	32.0
	5	2.05	4.17	6.40	8.75	11.2	13.9	16.6	19.3	22.2	25.0	27.9	30.9
	6	1.89	3.86	5.96	8.20	10.6	13.1	15.7	18.4	21.2	24.0	26.9	29.8
	7	1.75	3.59	5.57	7.70	9.99	12.4	14.9	17.5	20.2	23.0	25.8	28.7
	8	1.63	3.35	5.22	7.25	9.43	11.7	14.2	16.7	19.3	22.1	24.8	27.7
	9	1.52	3.13	4.90	6.83	8.91	11.1	13.5	15.9	18.5	21.2	23.9	26.7
	10	1.42	2.94	4.61	6.45	8.44	10.6	12.8	15.2	17.7	20.3	23.0	25.7
	12	1.25	2.60	4.11	5.78	7.60	9.58	11.7	14.0	16.3	18.8	21.3	23.9
	14	1.11	2.32	3.69	5.21	6.90	8.73	10.7	12.8	15.0	17.4	19.8	22.3
	16	0.99	2.09	3.34	4.74	6.29	8.00	9.85	11.8	13.9	16.1	18.5	20.9
	18	0.90	1.90	3.04	4.33	5.77	7.36	9.10	11.0	12.9	15.0	17.3	19.5
	20	0.81	1.73	2.79	3.98	5.33	6.81	8.44	10.2	12.1	14.1	16.2	18.4
	24	0.68	1.47	2.38	3.42	4.60	5.91	7.35	8.91	10.6	12.4	14.3	16.3
	28	0.59	1.28	2.08	2.99	4.03	5.20	6.49	7.90	9.42	11.1	12.8	14.6
	32	0.52	1.13	1.84	2.65	3.59	4.63	5.80	7.07	8.46	9.95	11.6	13.3
	36	0.46	1.01	1.65	2.38	3.23	4.17	5.23	6.40	7.67	9.04	10.5	12.1
6	2	2.64	5.38	8.22	11.1	14.1	17.0	20.0	23.0	25.9	28.9	31.9	34.8
	3	2.43	5.02	7.78	10.7	13.6	16.6	19.5	22.5	25.5	28.5	31.4	34.4
	4	2.23	4.67	7.33	10.2	13.1	16.0	19.0	22.0	25.0	28.0	31.0	33.9
	5	2.05	4.34	6.90	9.66	12.5	15.5	18.4	21.4	24.4	27.4	30.4	33.4
	6	1.89	4.06	6.50	9.19	12.0	14.9	17.9	20.9	23.9	26.9	29.9	32.9
	7	1.75	3.80	6.16	8.76	11.5	14.4	17.3	20.3	23.3	26.3	29.3	32.3
	8	1.63	3.57	5.84	8.36	11.1	13.9	16.8	19.7	22.7	25.7	28.7	31.7
	9	1.52	3.36	5.54	7.99	10.6	13.4	16.2	19.2	22.1	25.1	28.1	31.1
	10	1.42	3.17	5.27	7.63	10.2	12.9	15.7	18.6	21.5	24.5	27.5	30.5
	12	1.25	2.84	4.78	6.99	9.40	12.0	14.7	17.6	20.4	23.4	26.3	29.3
	14	1.11	2.57	4.36	6.42	8.70	11.2	13.8	16.6	19.4	22.3	25.2	28.2
	16	0.99	2.33	3.99	5.92	8.09	10.5	13.0	15.7	18.4	21.2	24.1	27.0
	18	0.90	2.13	3.68	5.49	7.54	9.80	12.2	14.8	17.5	20.3	23.1	26.0
	20	0.81	1.96	3.40	5.10	7.05	9.21	11.6	14.0	16.6	19.3	22.1	24.9
	24	0.68	1.68	2.95	4.46	6.22	8.19	10.4	12.7	15.1	17.7	20.3	23.0
	28	0.59	1.47	2.59	3.95	5.55	7.35	9.34	11.5	13.8	16.2	18.7	21.3
	32	0.52	1.31	2.31	3.54	4.99	6.65	8.49	10.5	12.7	14.9	17.3	19.8
	36	0.46	1.17	2.08	3.20	4.54	6.06	7.77	9.64	11.7	13.8	16.1	18.5

Table 7–12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

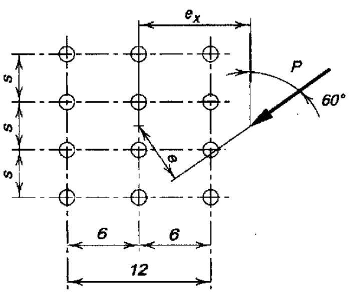
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	*e_x*, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.83	5.64	8.45	11.3	14.1	16.9	19.8	22.6	25.5	28.4	31.3	34.2
	3	2.72	5.43	8.13	10.8	13.6	16.3	19.1	21.9	24.8	27.6	30.5	33.4
	4	2.59	5.18	7.77	10.4	13.0	15.7	18.5	21.2	24.0	26.8	29.7	32.5
	5	2.46	4.92	7.40	9.92	12.5	15.1	17.8	20.5	23.2	26.0	28.9	31.7
	6	2.32	4.66	7.03	9.46	12.0	14.5	17.1	19.8	22.5	25.2	28.0	30.8
	7	2.19	4.41	6.68	9.02	11.4	13.9	16.5	19.1	21.8	24.5	27.2	30.0
	8	2.07	4.17	6.35	8.61	11.0	13.4	15.9	18.4	21.1	23.7	26.5	29.2
	9	1.95	3.95	6.04	8.22	10.5	12.9	15.3	17.8	20.4	23.0	25.7	28.5
	10	1.84	3.74	5.75	7.86	10.1	12.4	14.8	17.3	19.8	22.4	25.0	27.7
	12	1.65	3.38	5.22	7.19	9.28	11.5	13.8	16.2	18.6	21.1	23.7	26.3
	14	1.49	3.06	4.76	6.61	8.58	10.7	12.9	15.2	17.5	20.0	22.5	25.0
	16	1.35	2.79	4.37	6.09	7.95	9.93	12.0	14.2	16.5	18.9	21.3	23.8
	18	1.23	2.55	4.02	5.64	7.39	9.28	11.3	13.4	15.6	17.9	20.3	22.7
	20	1.12	2.35	3.72	5.24	6.90	8.69	10.6	12.6	14.8	17.0	19.3	21.7
	24	0.95	2.02	3.22	4.57	6.06	7.68	9.43	11.3	13.3	15.4	17.5	19.8
	28	0.83	1.76	2.84	4.04	5.39	6.86	8.47	10.2	12.0	14.0	16.0	18.1
	32	0.73	1.56	2.53	3.61	4.84	6.19	7.66	9.26	11.0	12.8	14.7	16.7
	36	0.65	1.40	2.27	3.26	4.38	5.62	6.98	8.46	10.1	11.7	13.5	15.4
6	2	2.83	5.64	8.47	11.3	14.2	17.1	20.0	23.0	25.9	28.9	31.8	34.8
	3	2.72	5.44	8.19	11.0	13.8	16.7	19.6	22.6	25.5	28.4	31.4	34.3
	4	2.59	5.21	7.88	10.6	13.4	16.3	19.2	22.1	25.0	28.0	30.9	33.9
	5	2.46	4.97	7.57	10.3	13.1	15.9	18.8	21.7	24.6	27.5	30.4	33.4
	6	2.32	4.73	7.27	9.91	12.7	15.5	18.3	21.2	24.1	27.0	30.0	32.9
	7	2.19	4.51	6.97	9.56	12.3	15.0	17.9	20.8	23.7	26.6	29.5	32.4
	8	2.07	4.29	6.69	9.23	11.9	14.6	17.5	20.3	23.2	26.1	29.0	32.0
	9	1.95	4.09	6.43	8.92	11.5	14.3	17.0	19.9	22.8	25.6	28.6	31.5
	10	1.84	3.90	6.18	8.63	11.2	13.9	16.6	19.5	22.3	25.2	28.1	31.0
	12	1.65	3.56	5.73	8.08	10.6	13.2	15.9	18.7	21.5	24.3	27.2	30.1
	14	1.49	3.27	5.32	7.59	10.0	12.6	15.2	17.9	20.7	23.5	26.3	29.2
	16	1.35	3.01	4.95	7.13	9.48	12.0	14.5	17.2	19.9	22.7	25.5	28.4
	18	1.23	2.78	4.63	6.71	8.98	11.4	13.9	16.5	19.2	22.0	24.7	27.6
	20	1.12	2.58	4.34	6.33	8.52	10.9	13.3	15.9	18.5	21.2	24.0	26.8
	24	0.95	2.25	3.84	5.67	7.70	9.91	12.3	14.7	17.3	19.9	22.6	25.3
	28	0.83	1.98	3.43	5.11	7.00	9.08	11.3	13.7	16.1	18.7	21.3	23.9
	32	0.73	1.77	3.09	4.64	6.40	8.36	10.5	12.7	15.1	17.5	20.1	22.6
	36	0.65	1.60	2.81	4.24	5.89	7.73	9.74	11.9	14.2	16.5	19.0	21.5

Table 7–12 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

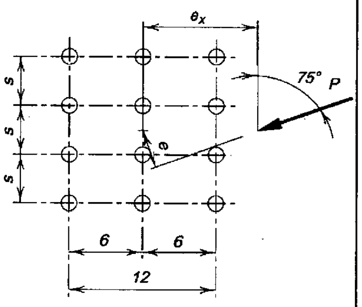
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.92	5.83	8.73	11.6	14.5	17.4	20.3	23.1	26.0	28.9	31.8	34.7
	3	2.89	5.77	8.63	11.5	14.3	17.2	20.0	22.8	25.7	28.5	31.4	34.2
	4	2.86	5.70	8.51	11.3	14.1	16.9	19.7	22.5	25.3	28.1	30.9	33.7
	5	2.82	5.61	8.38	11.1	13.9	16.6	19.4	22.1	24.9	27.7	30.5	33.3
	6	2.77	5.51	8.23	10.9	13.6	16.3	19.0	21.8	24.5	27.2	30.0	32.8
	7	2.72	5.40	8.06	10.7	13.4	16.0	18.7	21.4	24.1	26.8	29.6	32.3
	8	2.66	5.29	7.89	10.5	13.1	15.7	18.3	21.0	23.7	26.4	29.1	31.9
	9	2.60	5.16	7.71	10.3	12.8	15.4	18.0	20.6	23.3	26.0	28.7	31.4
	10	2.53	5.04	7.53	10.0	12.6	15.1	17.7	20.3	22.9	25.6	28.3	31.0
	12	2.40	4.78	7.16	9.57	12.0	14.5	17.0	19.6	22.1	24.8	27.4	30.1
	14	2.26	4.52	6.80	9.12	11.5	13.9	16.4	18.9	21.4	24.0	26.6	29.3
	16	2.13	4.27	6.45	8.68	11.0	13.3	15.8	18.2	20.7	23.3	25.9	28.5
	18	2.00	4.03	6.12	8.27	10.5	12.8	15.2	17.6	20.1	22.6	25.1	27.7
	20	1.89	3.81	5.80	7.88	10.1	12.3	14.6	17.0	19.4	21.9	24.4	27.0
	24	1.67	3.41	5.24	7.18	9.22	11.4	13.6	15.9	18.2	20.7	23.1	25.6
	28	1.49	3.06	4.75	6.56	8.49	10.5	12.6	14.9	17.1	19.5	21.9	24.3
	32	1.34	2.77	4.33	6.02	7.84	9.77	11.8	13.9	16.1	18.4	20.7	23.1
	36	1.21	2.52	3.97	5.56	7.27	9.10	11.1	13.1	15.2	17.4	19.7	22.0
6	2	2.92	5.82	8.71	11.6	14.5	17.4	20.3	23.5	26.4	29.3	32.3	35.2
	3	2.89	5.76	8.60	11.4	14.3	17.1	20.0	22.9	25.8	28.7	31.7	34.6
	4	2.86	5.68	8.47	11.3	14.1	16.9	19.8	22.6	25.5	28.4	31.3	34.2
	5	2.82	5.59	8.34	11.1	13.9	16.7	19.5	22.4	25.2	28.1	31.0	33.9
	6	2.77	5.49	8.19	10.9	13.7	16.4	19.2	22.1	24.9	27.8	30.7	33.6
	7	2.72	5.39	8.04	10.7	13.4	16.2	19.0	21.8	24.6	27.5	30.4	33.3
	8	2.66	5.27	7.89	10.5	13.2	16.0	18.8	21.6	24.4	27.2	30.1	33.0
	9	2.60	5.16	7.74	10.4	13.0	15.8	18.5	21.3	24.1	27.0	29.8	32.7
	10	2.53	5.04	7.58	10.2	12.8	15.5	18.3	21.0	23.9	26.7	29.5	32.4
	12	2.40	4.81	7.27	9.81	12.4	15.1	17.8	20.6	23.3	26.2	29.0	31.8
	14	2.26	4.57	6.97	9.47	12.0	14.7	17.4	20.1	22.9	25.6	28.4	31.3
	16	2.13	4.35	6.69	9.13	11.7	14.3	16.9	19.6	22.4	25.1	27.9	30.7
	18	2.00	4.13	6.41	8.82	11.3	13.9	16.5	19.2	21.9	24.7	27.4	30.2
	20	1.89	3.93	6.15	8.51	11.0	13.5	16.1	18.8	21.5	24.2	27.0	29.8
	24	1.67	3.57	5.67	7.95	10.4	12.9	15.4	18.0	20.7	23.4	26.1	28.8
	28	1.49	3.25	5.25	7.44	9.77	12.2	14.7	17.3	19.9	22.6	25.3	28.0
	32	1.34	2.97	4.87	6.98	9.23	11.6	14.1	16.6	19.2	21.8	24.5	27.2
	36	1.21	2.73	4.54	6.56	8.74	11.1	13.5	16.0	18.5	21.1	23.7	26.4

Table 7–13
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

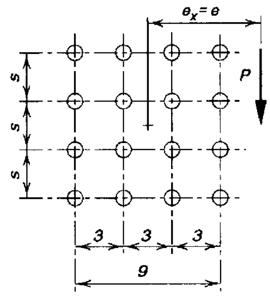
$\phi = 0.75$ $\quad\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.60	5.70	9.24	13.2	17.3	21.4	25.6	29.7	33.8	37.8	41.9	45.9
	3	2.23	4.92	8.05	11.7	15.6	19.7	23.9	28.1	32.3	36.4	40.6	44.7
	4	1.94	4.30	7.09	10.4	14.0	18.0	22.1	26.3	30.5	34.7	38.9	43.1
	5	1.69	3.79	6.30	9.29	12.6	16.4	20.3	24.4	28.6	32.9	37.1	41.4
	6	1.49	3.37	5.65	8.37	11.5	14.9	18.7	22.6	26.7	30.9	35.2	39.4
	7	1.32	3.03	5.10	7.59	10.4	13.7	17.2	21.0	24.9	29.0	33.2	37.5
	8	1.18	2.74	4.63	6.92	9.56	12.6	15.9	19.5	23.3	27.3	31.4	35.5
	9	1.07	2.50	4.24	6.35	8.81	11.6	14.7	18.1	21.7	25.6	29.6	33.7
	10	0.98	2.29	3.89	5.86	8.15	10.8	13.7	16.9	20.3	24.0	27.9	31.9
	12	0.83	1.96	3.34	5.06	7.06	9.37	12.0	14.8	17.9	21.3	24.9	28.6
	14	0.73	1.72	2.92	4.44	6.21	8.27	10.6	13.2	16.0	19.1	22.3	25.8
	16	0.65	1.52	2.59	3.95	5.54	7.39	9.48	11.8	14.4	17.2	20.2	23.4
	18	0.58	1.37	2.33	3.55	4.99	6.67	8.57	10.7	13.1	15.6	18.4	21.4
	20	0.53	1.24	2.11	3.23	4.53	6.07	7.81	9.77	11.9	14.3	16.9	19.6
	24	0.44	1.04	1.78	2.72	3.83	5.14	6.62	8.30	10.2	12.2	14.4	16.8
	28	0.38	0.90	1.54	2.35	3.31	4.45	5.73	7.20	8.82	10.6	12.6	14.7
	32	0.34	0.79	1.36	2.07	2.91	3.92	5.05	6.35	7.79	9.38	11.1	13.0
	36	0.30	0.71	1.21	1.85	2.60	3.50	4.51	5.68	6.96	8.39	9.95	11.6
	C′	11.3	26.0	44.7	68.1	96.0	129	167	210	258	312	371	435
6	2	2.60	6.48	10.7	14.8	18.9	23.0	27.0	31.0	34.9	38.9	42.9	46.8
	3	2.23	5.75	9.79	14.0	18.2	22.3	26.4	30.5	34.5	38.5	42.5	46.5
	4	1.94	5.12	8.91	13.1	17.4	21.6	25.7	29.9	33.9	38.0	42.0	46.1
	5	1.69	4.58	8.10	12.2	16.4	20.7	24.9	29.1	33.2	37.4	41.4	45.5
	6	1.49	4.13	7.37	11.3	15.5	19.7	24.0	28.3	32.5	36.6	40.8	44.9
	7	1.32	3.74	6.74	10.5	14.5	18.8	23.1	27.3	31.6	35.8	40.0	44.1
	8	1.18	3.41	6.20	9.73	13.6	17.8	22.1	26.4	30.6	34.9	39.1	43.3
	9	1.07	3.13	5.73	9.05	12.8	16.9	21.1	25.4	29.7	34.0	38.2	42.5
	10	0.98	2.89	5.31	8.45	12.0	16.0	20.1	24.4	28.7	33.0	37.3	41.5
	12	0.83	2.50	4.63	7.43	10.7	14.3	18.3	22.4	26.7	31.0	35.3	39.6
	14	0.73	2.19	4.09	6.60	9.53	12.9	16.7	20.6	24.7	29.0	33.3	37.6
	16	0.65	1.95	3.65	5.93	8.59	11.7	15.2	19.0	22.9	27.1	31.3	35.5
	18	0.58	1.76	3.29	5.37	7.81	10.7	14.0	17.5	21.3	25.3	29.4	33.6
	20	0.53	1.60	2.99	4.90	7.15	9.85	12.9	16.2	19.8	23.6	27.6	31.7
	24	0.44	1.35	2.53	4.16	6.10	8.44	11.1	14.0	17.3	20.8	24.4	28.3
	28	0.38	1.17	2.19	3.61	5.31	7.37	9.69	12.3	15.2	18.4	21.8	25.3
	32	0.34	1.03	1.93	3.19	4.69	6.53	8.61	11.0	13.6	16.5	19.6	22.9
	36	0.30	0.92	1.72	2.85	4.20	5.85	7.73	9.89	12.3	14.9	17.7	20.8
	C′	11.3	33.7	63.7	106	156	219	291	375	469	574	690	817

Table 7–13 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

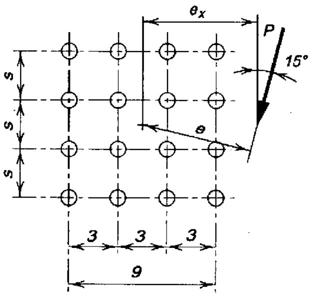
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.68	5.77	9.31	13.2	17.2	21.3	25.4	29.5	33.6	37.6	41.7	45.7
	3	2.30	5.00	8.17	11.7	15.6	19.6	23.7	27.8	32.0	36.1	40.2	44.3
	4	1.99	4.38	7.22	10.4	14.1	17.9	21.9	26.0	30.2	34.4	38.5	42.7
	5	1.74	3.88	6.43	9.37	12.7	16.4	20.2	24.2	28.3	32.5	36.7	40.9
	6	1.53	3.45	5.77	8.47	11.6	15.0	18.6	22.5	26.5	30.6	34.8	39.0
	7	1.36	3.10	5.21	7.71	10.6	13.7	17.2	20.9	24.8	28.8	32.9	37.1
	8	1.22	2.81	4.74	7.05	9.70	12.7	15.9	19.5	23.2	27.1	31.1	35.2
	9	1.11	2.57	4.34	6.48	8.95	11.7	14.8	18.1	21.7	25.5	29.4	33.4
	10	1.01	2.36	4.00	5.98	8.29	10.9	13.8	17.0	20.4	24.0	27.7	31.6
	12	0.86	2.02	3.44	5.18	7.21	9.52	12.1	15.0	18.1	21.4	24.9	28.5
	14	0.75	1.77	3.01	4.55	6.36	8.43	10.8	13.3	16.1	19.2	22.4	25.8
	16	0.67	1.57	2.68	4.05	5.67	7.54	9.66	12.0	14.6	17.3	20.3	23.5
	18	0.60	1.41	2.40	3.65	5.12	6.81	8.74	10.9	13.3	15.8	18.6	21.5
	20	0.54	1.28	2.18	3.32	4.66	6.21	7.98	9.95	12.1	14.5	17.1	19.8
	24	0.46	1.08	1.84	2.80	3.94	5.26	6.78	8.47	10.4	12.4	14.6	17.0
	28	0.40	0.93	1.59	2.43	3.41	4.56	5.89	7.37	9.02	10.8	12.8	14.9
	32	0.35	0.82	1.40	2.14	3.00	4.03	5.19	6.51	7.98	9.59	11.3	13.2
	36	0.31	0.73	1.25	1.91	2.68	3.60	4.65	5.83	7.15	8.59	10.2	11.9
6	2	2.68	6.48	10.6	14.7	18.8	22.9	26.9	30.9	34.8	38.8	42.8	46.7
	3	2.30	5.75	9.75	13.9	18.1	22.2	26.3	30.3	34.3	38.3	42.3	46.3
	4	1.99	5.13	8.91	13.0	17.2	21.4	25.5	29.6	33.7	37.7	41.8	45.8
	5	1.74	4.61	8.14	12.1	16.3	20.5	24.7	28.8	33.0	37.1	41.1	45.2
	6	1.53	4.17	7.45	11.2	15.3	19.5	23.7	27.9	32.1	36.3	40.4	44.5
	7	1.36	3.79	6.84	10.4	14.4	18.6	22.8	27.0	31.2	35.4	39.6	43.7
	8	1.22	3.46	6.30	9.71	13.6	17.6	21.8	26.0	30.3	34.5	38.7	42.9
	9	1.11	3.19	5.83	9.05	12.8	16.7	20.9	25.1	29.3	33.5	37.8	42.0
	10	1.01	2.94	5.42	8.47	12.0	15.9	19.9	24.1	28.3	32.6	36.8	41.0
	12	0.86	2.55	4.73	7.47	10.7	14.3	18.2	22.2	26.4	30.6	34.8	39.1
	14	0.75	2.24	4.18	6.66	9.62	12.9	16.6	20.5	24.5	28.6	32.8	37.1
	16	0.67	2.00	3.74	6.00	8.71	11.8	15.2	18.9	22.8	26.8	30.9	35.1
	18	0.60	1.80	3.38	5.45	7.94	10.8	14.0	17.5	21.2	25.1	29.1	33.2
	20	0.54	1.64	3.08	4.98	7.28	9.92	13.0	16.2	19.8	23.5	27.4	31.4
	24	0.46	1.39	2.60	4.25	6.23	8.54	11.2	14.1	17.3	20.8	24.4	28.1
	28	0.40	1.20	2.26	3.69	5.43	7.48	9.85	12.5	15.4	18.5	21.8	25.3
	32	0.35	1.06	1.99	3.26	4.81	6.65	8.77	11.1	13.8	16.6	19.7	22.9
	36	0.31	0.94	1.78	2.92	4.31	5.97	7.89	10.0	12.5	15.1	17.9	20.9

Table 7-13 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

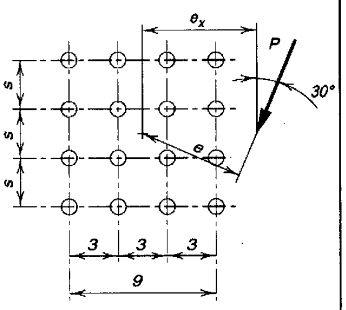
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.90	6.06	9.59	13.4	17.3	21.3	25.3	29.4	33.4	37.4	41.4	45.4
	3	2.50	5.31	8.52	12.1	15.8	19.7	23.7	27.8	31.8	35.9	40.0	44.0
	4	2.18	4.70	7.62	10.9	14.4	18.2	22.1	26.1	30.1	34.2	38.3	42.4
	5	1.91	4.18	6.85	9.86	13.2	16.8	20.5	24.4	28.4	32.5	36.6	40.7
	6	1.69	3.75	6.19	8.98	12.1	15.5	19.1	22.9	26.8	30.7	34.8	38.9
	7	1.51	3.38	5.63	8.21	11.1	14.3	17.8	21.4	25.2	29.1	33.1	37.1
	8	1.36	3.07	5.14	7.55	10.3	13.3	16.6	20.0	23.7	27.5	31.4	35.4
	9	1.23	2.81	4.73	6.97	9.54	12.4	15.5	18.8	22.3	26.0	29.8	33.7
	10	1.13	2.59	4.37	6.46	8.88	11.6	14.5	17.7	21.1	24.7	28.3	32.2
	12	0.96	2.23	3.78	5.62	7.78	10.2	12.9	15.8	18.9	22.2	25.7	29.3
	14	0.84	1.95	3.32	4.96	6.90	9.08	11.5	14.2	17.1	20.1	23.4	26.8
	16	0.74	1.73	2.96	4.43	6.19	8.17	10.4	12.9	15.5	18.4	21.4	24.6
	18	0.67	1.56	2.66	4.00	5.60	7.41	9.46	11.7	14.2	16.8	19.7	22.7
	20	0.61	1.42	2.42	3.65	5.11	6.77	8.67	10.8	13.1	15.5	18.2	21.0
	24	0.51	1.20	2.04	3.09	4.34	5.77	7.41	9.22	11.2	13.4	15.7	18.2
	28	0.44	1.03	1.77	2.68	3.77	5.01	6.46	8.05	9.83	11.8	13.9	16.1
	32	0.39	0.91	1.56	2.36	3.32	4.43	5.71	7.14	8.72	10.5	12.3	14.4
	36	0.35	0.81	1.39	2.11	2.97	3.97	5.12	6.40	7.84	9.41	11.1	13.0
6	2	2.90	6.59	10.6	14.7	18.7	22.7	26.7	30.7	34.7	38.7	42.6	46.6
	3	2.50	5.88	9.83	13.9	18.0	22.0	26.1	30.1	34.1	38.1	42.1	46.1
	4	2.18	5.30	9.05	13.0	17.1	21.2	25.3	29.4	33.5	37.5	41.5	45.5
	5	1.91	4.81	8.35	12.2	16.3	20.4	24.5	28.6	32.7	36.8	40.8	44.9
	6	1.69	4.38	7.72	11.4	15.4	19.5	23.6	27.7	31.8	35.9	40.0	44.1
	7	1.51	4.01	7.15	10.7	14.6	18.6	22.7	26.8	31.0	35.1	39.2	43.3
	8	1.36	3.69	6.64	10.0	13.8	17.7	21.8	25.9	30.0	34.2	38.3	42.4
	9	1.23	3.41	6.19	9.41	13.0	16.9	20.9	25.0	29.1	33.3	37.4	41.6
	10	1.13	3.16	5.79	8.85	12.4	16.1	20.1	24.1	28.2	32.4	36.5	40.6
	12	0.96	2.76	5.09	7.88	11.1	14.7	18.5	22.4	26.4	30.5	34.6	38.8
	14	0.84	2.44	4.54	7.08	10.1	13.4	17.0	20.8	24.7	28.8	32.8	36.9
	16	0.74	2.18	4.08	6.41	9.21	12.3	15.7	19.4	23.2	27.1	31.1	35.1
	18	0.67	1.97	3.70	5.85	8.45	11.4	14.6	18.1	21.7	25.5	29.4	33.4
	20	0.61	1.80	3.38	5.37	7.80	10.5	13.6	16.9	20.4	24.1	27.9	31.8
	24	0.51	1.53	2.87	4.61	6.74	9.16	11.9	14.9	18.1	21.5	25.1	28.8
	28	0.44	1.32	2.49	4.02	5.91	8.07	10.5	13.3	16.2	19.4	22.7	26.2
	32	0.39	1.17	2.20	3.57	5.26	7.20	9.45	11.9	14.6	17.6	20.7	23.9
	36	0.35	1.05	1.97	3.21	4.73	6.49	8.55	10.8	13.3	16.0	18.9	22.0

Table 7-13 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ or $\Omega = 2.00$

LRFD	ASD
$C_{min} = \frac{P_u}{\phi r_n}$	$C_{min} = \frac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

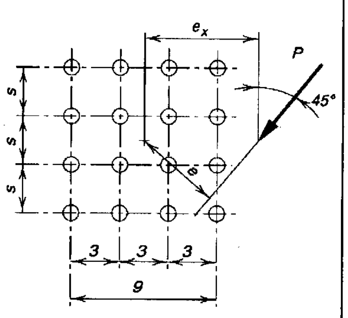
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.26	6.62	10.2	13.9	17.7	21.5	25.5	29.4	33.4	37.3	41.3	45.3
	3	2.87	5.92	9.19	12.7	16.4	20.2	24.0	28.0	31.9	35.9	39.9	43.9
	4	2.54	5.31	8.36	11.7	15.2	18.8	22.6	26.5	30.4	34.4	38.4	42.4
	5	2.25	4.78	7.63	10.8	14.1	17.6	21.3	25.1	29.0	32.9	36.8	40.8
	6	2.01	4.33	6.99	9.94	13.1	16.5	20.1	23.8	27.5	31.4	35.3	39.3
	7	1.81	3.93	6.42	9.20	12.2	15.5	18.9	22.5	26.2	30.0	33.8	37.7
	8	1.64	3.60	5.92	8.55	11.4	14.6	17.9	21.3	24.9	28.6	32.4	36.3
	9	1.49	3.31	5.49	7.96	10.7	13.7	16.9	20.3	23.8	27.4	31.1	34.9
	10	1.37	3.06	5.10	7.44	10.1	12.9	16.0	19.2	22.7	26.2	29.8	33.6
	12	1.17	2.65	4.46	6.55	8.93	11.6	14.4	17.5	20.7	24.0	27.5	31.1
	14	1.03	2.33	3.95	5.83	8.00	10.4	13.1	15.9	18.9	22.1	25.4	28.8
	16	0.91	2.08	3.54	5.24	7.23	9.47	11.9	14.6	17.4	20.4	23.6	26.8
	18	0.82	1.88	3.20	4.75	6.59	8.66	10.9	13.4	16.1	18.9	21.9	25.0
	20	0.74	1.71	2.92	4.35	6.04	7.96	10.1	12.4	15.0	17.6	20.5	23.5
	24	0.63	1.45	2.48	3.71	5.18	6.84	8.71	10.8	13.0	15.4	18.0	20.7
	28	0.54	1.26	2.15	3.23	4.52	5.99	7.65	9.50	11.5	13.7	16.0	18.5
	32	0.48	1.11	1.90	2.86	4.00	5.31	6.81	8.48	10.3	12.3	14.4	16.7
	36	0.43	0.99	1.69	2.56	3.59	4.77	6.13	7.64	9.30	11.1	13.1	15.2
6	2	3.26	6.89	10.8	14.7	18.7	22.7	26.6	30.6	34.6	38.5	42.5	46.5
	3	2.87	6.28	10.1	14.0	18.0	22.0	26.0	30.0	33.9	37.9	41.9	45.9
	4	2.54	5.74	9.38	13.3	17.2	21.2	25.2	29.2	33.2	37.2	41.2	45.2
	5	2.25	5.27	8.75	12.6	16.5	20.4	24.5	28.5	32.5	36.5	40.5	44.5
	6	2.01	4.85	8.20	11.9	15.7	19.7	23.7	27.7	31.7	35.7	39.7	43.8
	7	1.81	4.49	7.70	11.3	15.0	18.9	22.9	26.9	30.9	34.9	39.0	43.0
	8	1.64	4.16	7.25	10.7	14.4	18.2	22.1	26.1	30.1	34.1	38.2	42.2
	9	1.49	3.87	6.83	10.2	13.7	17.5	21.4	25.3	29.3	33.3	37.4	41.4
	10	1.37	3.62	6.45	9.65	13.1	16.8	20.7	24.6	28.5	32.5	36.6	40.6
	12	1.17	3.19	5.78	8.75	12.0	15.6	19.3	23.1	27.0	31.0	35.0	39.0
	14	1.03	2.84	5.21	7.97	11.1	14.5	18.1	21.8	25.6	29.5	33.4	37.4
	16	0.91	2.56	4.74	7.30	10.2	13.5	16.9	20.5	24.3	28.1	32.0	35.9
	18	0.82	2.33	4.33	6.72	9.48	12.6	15.9	19.4	23.0	26.7	30.6	34.4
	20	0.74	2.13	3.98	6.21	8.83	11.8	15.0	18.3	21.8	25.5	29.2	33.1
	24	0.63	1.82	3.42	5.38	7.74	10.4	13.3	16.5	19.8	23.2	26.8	30.5
	28	0.54	1.59	2.99	4.74	6.87	9.30	12.0	14.9	18.0	21.3	24.7	28.2
	32	0.48	1.41	2.65	4.22	6.17	8.38	10.8	13.6	16.5	19.5	22.8	26.1
	36	0.43	1.26	2.38	3.81	5.59	7.62	9.89	12.4	15.2	18.0	21.1	24.3

Table 7–13 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

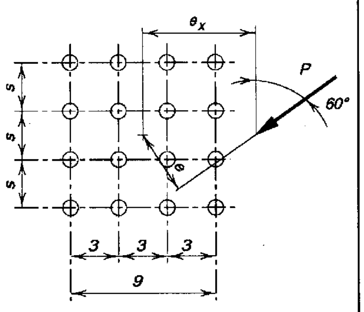
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.63	7.25	10.9	14.6	18.3	22.1	25.9	29.7	33.6	37.5	41.4	45.3
	3	3.38	6.77	10.2	13.8	17.4	21.1	24.8	28.6	32.4	36.3	40.2	44.1
	4	3.10	6.27	9.55	13.0	16.5	20.1	23.7	27.5	31.3	35.1	38.9	42.8
	5	2.84	5.80	8.92	12.2	15.6	19.1	22.7	26.4	30.1	33.9	37.8	41.6
	6	2.60	5.36	8.33	11.5	14.8	18.2	21.7	25.4	29.1	32.8	36.6	40.4
	7	2.38	4.96	7.79	10.8	14.1	17.4	20.9	24.4	28.0	31.7	35.5	39.3
	8	2.19	4.60	7.30	10.2	13.4	16.6	20.0	23.5	27.1	30.7	34.4	38.2
	9	2.02	4.28	6.85	9.68	12.7	15.9	19.2	22.6	26.1	29.7	33.4	37.1
	10	1.87	3.99	6.45	9.17	12.1	15.2	18.4	21.8	25.3	28.8	32.4	36.1
	12	1.62	3.51	5.75	8.27	11.0	13.9	17.0	20.3	23.6	27.0	30.6	34.1
	14	1.43	3.12	5.18	7.50	10.1	12.8	15.8	18.9	22.1	25.4	28.9	32.4
	16	1.27	2.81	4.70	6.85	9.23	11.9	14.7	17.6	20.7	24.0	27.3	30.7
	18	1.15	2.56	4.29	6.28	8.52	11.0	13.7	16.5	19.5	22.6	25.8	29.1
	20	1.04	2.34	3.95	5.80	7.89	10.2	12.8	15.5	18.4	21.4	24.5	27.7
	24	0.88	2.00	3.39	5.01	6.87	8.98	11.3	13.8	16.4	19.2	22.1	25.2
	28	0.76	1.74	2.96	4.39	6.07	7.97	10.1	12.3	14.8	17.4	20.1	23.0
	32	0.67	1.54	2.63	3.91	5.43	7.15	9.06	11.2	13.4	15.8	18.4	21.1
	36	0.60	1.38	2.36	3.52	4.91	6.48	8.22	10.2	12.3	14.5	16.9	19.4
6	2	3.63	7.29	11.1	14.9	18.8	22.7	26.6	30.5	34.5	38.4	42.4	46.3
	3	3.38	6.88	10.6	14.3	18.2	22.1	26.0	29.9	33.9	37.8	41.8	45.7
	4	3.10	6.46	10.0	13.8	17.6	21.5	25.4	29.3	33.2	37.2	41.1	45.1
	5	2.84	6.06	9.55	13.2	17.0	20.9	24.7	28.7	32.6	36.5	40.4	44.4
	6	2.60	5.69	9.09	12.7	16.4	20.3	24.1	28.0	31.9	35.9	39.8	43.8
	7	2.38	5.34	8.66	12.2	15.9	19.7	23.5	27.4	31.3	35.2	39.2	43.1
	8	2.19	5.03	8.27	11.7	15.4	19.1	22.9	26.8	30.7	34.6	38.5	42.4
	9	2.02	4.74	7.90	11.3	14.9	18.6	22.4	26.2	30.1	34.0	37.9	41.8
	10	1.87	4.47	7.55	10.9	14.4	18.1	21.8	25.6	29.5	33.4	37.3	41.2
	12	1.62	4.01	6.93	10.1	13.6	17.1	20.8	24.5	28.3	32.2	36.0	39.9
	14	1.43	3.63	6.38	9.46	12.8	16.2	19.8	23.5	27.3	31.0	34.9	38.7
	16	1.27	3.31	5.91	8.84	12.0	15.4	18.9	22.5	26.2	30.0	33.8	37.6
	18	1.15	3.04	5.49	8.28	11.3	14.6	18.0	21.6	25.2	28.9	32.7	36.5
	20	1.04	2.81	5.12	7.77	10.7	13.9	17.2	20.7	24.3	28.0	31.7	35.4
	24	0.88	2.44	4.49	6.90	9.62	12.6	15.8	19.1	22.6	26.1	29.8	33.4
	28	0.76	2.15	3.99	6.18	8.70	11.5	14.5	17.7	21.1	24.5	28.0	31.6
	32	0.67	1.91	3.58	5.58	7.93	10.6	13.4	16.5	19.7	23.0	26.4	29.9
	36	0.60	1.73	3.24	5.08	7.27	9.76	12.5	15.4	18.4	21.6	24.9	28.3

Table 7–13 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

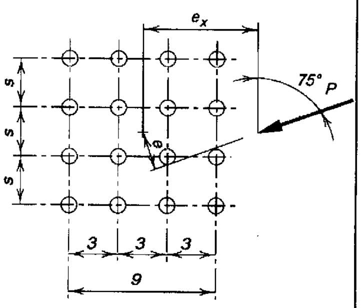
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.86	7.69	11.5	15.3	19.1	22.9	26.7	30.5	34.3	38.2	42.0	45.9
	3	3.79	7.53	11.2	14.9	18.6	22.4	26.1	29.9	33.6	37.4	41.3	45.1
	4	3.70	7.34	11.0	14.6	18.2	21.8	25.5	29.2	33.0	36.7	40.5	44.3
	5	3.59	7.13	10.6	14.2	17.7	21.3	24.9	28.6	32.3	36.1	39.8	43.6
	6	3.47	6.89	10.3	13.8	17.2	20.8	24.4	28.0	31.7	35.4	39.1	42.9
	7	3.34	6.65	9.98	13.4	16.8	20.3	23.8	27.4	31.1	34.7	38.4	42.2
	8	3.20	6.40	9.64	12.9	16.3	19.8	23.3	26.8	30.4	34.1	37.8	41.5
	9	3.07	6.16	9.31	12.6	15.9	19.3	22.8	26.3	29.9	33.5	37.1	40.8
	10	2.94	5.91	8.98	12.2	15.4	18.8	22.2	25.7	29.3	32.9	36.5	40.2
	12	2.68	5.45	8.36	11.4	14.6	17.9	21.3	24.7	28.2	31.8	35.4	39.0
	14	2.45	5.03	7.79	10.7	13.8	17.1	20.4	23.8	27.2	30.7	34.3	37.9
	16	2.24	4.65	7.28	10.1	13.1	16.3	19.5	22.9	26.3	29.7	33.2	36.8
	18	2.06	4.31	6.81	9.55	12.5	15.5	18.7	22.0	25.4	28.8	32.2	35.8
	20	1.90	4.01	6.40	9.03	11.9	14.8	18.0	21.2	24.5	27.9	31.3	34.8
	24	1.63	3.51	5.69	8.13	10.8	13.6	16.6	19.7	22.8	26.1	29.5	32.9
	28	1.43	3.11	5.11	7.36	9.83	12.5	15.3	18.3	21.4	24.6	27.8	31.1
	32	1.27	2.79	4.62	6.71	9.02	11.5	14.2	17.1	20.0	23.1	26.3	29.5
	36	1.14	2.53	4.22	6.15	8.31	10.7	13.3	16.0	18.8	21.8	24.9	28.0
6	2	3.86	7.67	11.5	15.3	19.1	23.0	26.9	30.8	35.2	39.1	43.0	47.0
	3	3.79	7.51	11.2	15.0	18.8	22.6	26.4	30.3	34.2	38.1	42.1	46.0
	4	3.70	7.32	11.0	14.7	18.4	22.2	26.0	29.9	33.8	37.7	41.6	45.5
	5	3.59	7.12	10.7	14.4	18.1	21.8	25.6	29.5	33.3	37.2	41.1	45.0
	6	3.47	6.92	10.4	14.1	17.7	21.5	25.3	29.1	32.9	36.8	40.7	44.6
	7	3.34	6.70	10.2	13.8	17.4	21.1	24.9	28.7	32.5	36.4	40.2	44.1
	8	3.20	6.49	9.92	13.5	17.1	20.8	24.5	28.3	32.1	36.0	39.8	43.7
	9	3.07	6.28	9.66	13.2	16.8	20.5	24.2	28.0	31.8	35.6	39.4	43.3
	10	2.94	6.08	9.42	12.9	16.5	20.2	23.9	27.6	31.4	35.2	39.0	42.9
	12	2.68	5.69	8.95	12.4	15.9	19.5	23.2	26.9	30.7	34.5	38.3	42.1
	14	2.45	5.33	8.51	11.9	15.4	19.0	22.6	26.3	30.0	33.8	37.6	41.4
	16	2.24	4.99	8.10	11.4	14.9	18.4	22.0	25.7	29.4	33.1	36.9	40.7
	18	2.06	4.69	7.72	11.0	14.4	17.9	21.5	25.1	28.8	32.5	36.2	40.0
	20	1.90	4.42	7.36	10.6	13.9	17.4	21.0	24.6	28.2	31.9	35.6	39.3
	24	1.63	3.95	6.74	9.83	13.1	16.5	20.0	23.5	27.1	30.7	34.4	38.1
	28	1.43	3.57	6.21	9.16	12.3	15.6	19.0	22.5	26.1	29.7	33.3	36.9
	32	1.27	3.25	5.74	8.56	11.6	14.8	18.2	21.6	25.1	28.6	32.2	35.9
	36	1.14	2.98	5.33	8.02	11.0	14.1	17.3	20.7	24.1	27.6	31.2	34.8

Table 7–14
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 0°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

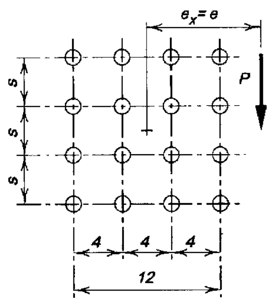
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.82	5.98	9.46	13.3	17.3	21.3	25.5	29.6	33.7	37.7	41.8	45.8
	3	2.50	5.31	8.43	12.0	15.7	19.7	23.8	28.0	32.2	36.3	40.4	44.6
	4	2.23	4.74	7.58	10.8	14.3	18.2	22.2	26.3	30.4	34.6	38.8	43.0
	5	2.01	4.27	6.86	9.82	13.1	16.7	20.5	24.5	28.6	32.8	37.0	41.3
	6	1.81	3.86	6.24	8.96	12.0	15.4	19.0	22.9	26.9	31.0	35.2	39.4
	7	1.64	3.52	5.70	8.22	11.1	14.2	17.6	21.3	25.2	29.2	33.3	37.5
	8	1.49	3.22	5.24	7.57	10.2	13.2	16.4	19.9	23.6	27.5	31.5	35.6
	9	1.36	2.96	4.83	7.01	9.48	12.3	15.3	18.6	22.1	25.9	29.8	33.8
	10	1.25	2.73	4.47	6.51	8.83	11.4	14.3	17.5	20.8	24.4	28.2	32.1
	12	1.07	2.37	3.89	5.68	7.74	10.1	12.6	15.5	18.5	21.8	25.3	29.0
	14	0.94	2.08	3.42	5.02	6.86	8.95	11.3	13.8	16.6	19.6	22.8	26.2
	16	0.83	1.86	3.05	4.49	6.15	8.04	10.2	12.5	15.0	17.8	20.7	23.9
	18	0.75	1.67	2.75	4.06	5.56	7.29	9.22	11.4	13.7	16.3	19.0	21.9
	20	0.68	1.52	2.50	3.70	5.07	6.65	8.43	10.4	12.6	14.9	17.5	20.2
	24	0.58	1.29	2.12	3.14	4.30	5.66	7.18	8.88	10.8	12.8	15.0	17.4
	28	0.50	1.12	1.84	2.72	3.73	4.92	6.24	7.73	9.37	11.2	13.1	15.2
	32	0.44	0.98	1.62	2.40	3.30	4.34	5.51	6.84	8.29	9.90	11.6	13.5
	36	0.40	0.88	1.45	2.15	2.95	3.89	4.94	6.13	7.43	8.88	10.4	12.1
	C′	15.0	32.8	54.2	79.9	110	145	184	229	279	333	393	458
6	2	2.82	6.54	10.6	14.8	18.9	22.9	26.9	30.9	34.9	38.9	42.8	46.8
	3	2.50	5.90	9.81	14.0	18.1	22.3	26.4	30.4	34.5	38.5	42.5	46.5
	4	2.23	5.33	9.01	13.1	17.3	21.5	25.7	29.8	33.9	37.9	42.0	46.0
	5	2.01	4.84	8.27	12.2	16.4	20.6	24.8	29.0	33.2	37.3	41.4	45.5
	6	1.81	4.42	7.60	11.4	15.5	19.7	24.0	28.2	32.4	36.6	40.7	44.8
	7	1.64	4.05	7.02	10.6	14.6	18.8	23.0	27.3	31.5	35.7	39.9	44.1
	8	1.49	3.73	6.51	9.94	13.7	17.8	22.0	26.3	30.6	34.8	39.1	43.3
	9	1.36	3.45	6.06	9.30	13.0	16.9	21.1	25.3	29.6	33.9	38.2	42.4
	10	1.25	3.20	5.66	8.72	12.2	16.1	20.2	24.4	28.6	32.9	37.2	41.5
	12	1.07	2.80	4.98	7.73	10.9	14.5	18.4	22.5	26.7	30.9	35.2	39.5
	14	0.94	2.47	4.43	6.92	9.81	13.2	16.8	20.7	24.8	29.0	33.2	37.5
	16	0.83	2.21	3.98	6.25	8.90	12.0	15.4	19.1	23.0	27.1	31.3	35.5
	18	0.75	2.00	3.60	5.68	8.13	11.0	14.2	17.7	21.4	25.3	29.4	33.6
	20	0.68	1.82	3.29	5.21	7.47	10.1	13.1	16.4	20.0	23.7	27.7	31.7
	24	0.58	1.55	2.79	4.45	6.40	8.72	11.3	14.3	17.5	20.9	24.5	28.3
	28	0.50	1.34	2.42	3.87	5.59	7.64	9.96	12.6	15.5	18.6	21.9	25.5
	32	0.44	1.18	2.14	3.43	4.95	6.79	8.87	11.2	13.8	16.7	19.7	23.0
	36	0.40	1.06	1.92	3.07	4.44	6.10	7.98	10.1	12.5	15.1	17.9	20.9
	C′	15.0	39.4	71.8	115	167	230	304	388	483	588	705	832

Table 7-14 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 15°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

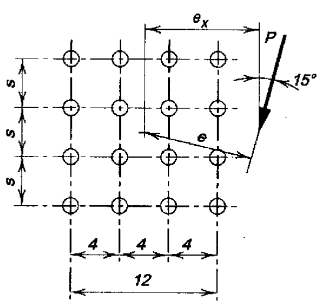
$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips
r_n = nominal strength per bolt, kips
e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)
e_x = horizontal component of e, in.
s = bolt spacing, in.
C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	2.91	6.06	9.56	13.3	17.2	21.3	25.3	29.4	33.5	37.5	41.6	45.6
	3	2.57	5.40	8.57	12.0	15.8	19.7	23.7	27.8	31.9	36.1	40.2	44.3
	4	2.30	4.84	7.72	10.9	14.4	18.2	22.1	26.1	30.2	34.3	38.5	42.6
	5	2.06	4.37	6.99	9.93	13.2	16.7	20.5	24.4	28.5	32.6	36.7	40.9
	6	1.86	3.96	6.37	9.09	12.1	15.5	19.0	22.8	26.7	30.8	34.9	39.0
	7	1.69	3.61	5.83	8.36	11.2	14.3	17.7	21.3	25.1	29.0	33.1	37.2
	8	1.53	3.31	5.36	7.72	10.4	13.3	16.5	19.9	23.6	27.4	31.3	35.3
	9	1.40	3.04	4.95	7.15	9.64	12.4	15.4	18.7	22.2	25.8	29.7	33.6
	10	1.29	2.81	4.59	6.65	9.0	11.6	14.5	17.6	20.9	24.4	28.1	31.9
	12	1.11	2.44	4.00	5.82	7.9	10.2	12.8	15.6	18.7	21.9	25.3	28.9
	14	0.97	2.15	3.52	5.15	7.0	9.12	11.5	14.0	16.8	19.8	22.9	26.3
	16	0.86	1.92	3.15	4.61	6.3	8.21	10.3	12.7	15.2	18.0	20.9	24.0
	18	0.78	1.73	2.84	4.17	5.7	7.45	9.41	11.6	13.9	16.5	19.2	22.1
	20	0.71	1.57	2.59	3.80	5.2	6.81	8.61	10.6	12.8	15.2	17.7	20.4
	24	0.60	1.33	2.19	3.23	4.4	5.80	7.36	9.07	11.0	13.0	15.3	17.6
	28	0.52	1.15	1.90	2.80	3.9	5.05	6.41	7.91	9.59	11.4	13.4	15.5
	32	0.46	1.02	1.68	2.48	3.4	4.46	5.67	7.01	8.50	10.1	11.9	13.8
	36	0.41	0.91	1.50	2.22	3.0	4.00	5.08	6.29	7.63	9.09	10.7	12.4
6	2	2.91	6.57	10.6	14.7	18.8	22.8	26.8	30.8	34.8	38.8	42.7	46.7
	3	2.57	5.93	9.81	13.9	18.0	22.1	26.2	30.3	34.3	38.3	42.3	46.3
	4	2.30	5.37	9.04	13.0	17.2	21.3	25.5	29.6	33.6	37.7	41.7	45.8
	5	2.06	4.89	8.33	12.2	16.3	20.5	24.6	28.8	32.9	37.0	41.1	45.1
	6	1.86	4.48	7.70	11.4	15.4	19.5	23.7	27.9	32.1	36.2	40.3	44.4
	7	1.69	4.12	7.13	10.6	14.5	18.6	22.8	27.0	31.2	35.4	39.5	43.7
	8	1.53	3.80	6.62	9.95	13.7	17.7	21.8	26.0	30.2	34.4	38.6	42.8
	9	1.40	3.52	6.17	9.32	12.9	16.8	20.9	25.1	29.3	33.5	37.7	41.9
	10	1.29	3.27	5.77	8.76	12.2	16.0	20.0	24.1	28.3	32.5	36.8	41.0
	12	1.11	2.86	5.09	7.80	11.0	14.5	18.3	22.3	26.4	30.6	34.8	39.0
	14	0.97	2.54	4.53	7.00	9.92	13.2	16.8	20.6	24.6	28.7	32.8	37.1
	16	0.86	2.27	4.08	6.34	9.02	12.0	15.4	19.0	22.9	26.9	30.9	35.1
	18	0.78	2.06	3.70	5.78	8.26	11.1	14.2	17.7	21.3	25.2	29.1	33.2
	20	0.71	1.88	3.38	5.30	7.60	10.2	13.2	16.4	19.9	23.6	27.5	31.4
	24	0.60	1.59	2.88	4.54	6.54	8.84	11.5	14.4	17.5	20.9	24.5	28.2
	28	0.52	1.38	2.50	3.96	5.72	7.77	10.1	12.7	15.6	18.7	22.0	25.4
	32	0.46	1.22	2.21	3.51	5.08	6.92	9.03	11.4	14.0	16.8	19.9	23.1
	36	0.41	1.09	1.98	3.15	4.56	6.23	8.15	10.3	12.7	15.3	18.1	21.1

Table 7–14 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 30°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

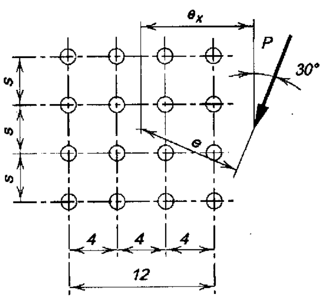
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, n											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.14	6.41	9.91	13.6	17.5	21.4	25.4	29.4	33.4	37.4	41.4	45.4
	3	2.79	5.75	8.95	12.4	16.1	20.0	23.9	27.9	31.9	35.9	40.0	44.0
	4	2.50	5.19	8.16	11.4	14.9	18.5	22.4	26.3	30.3	34.3	38.4	42.4
	5	2.25	4.71	7.45	10.5	13.7	17.2	20.9	24.7	28.6	32.6	36.7	40.7
	6	2.04	4.29	6.83	9.65	12.7	16.0	19.6	23.3	27.1	31.0	35.0	39.0
	7	1.85	3.93	6.28	8.92	11.8	15.0	18.3	21.9	25.6	29.4	33.3	37.3
	8	1.69	3.61	5.80	8.27	11.0	14.0	17.2	20.6	24.2	27.9	31.7	35.6
	9	1.55	3.33	5.38	7.70	10.3	13.1	16.2	19.4	22.9	26.5	30.2	34.0
	10	1.43	3.08	5.00	7.19	9.64	12.3	15.3	18.4	21.7	25.2	28.8	32.5
	12	1.23	2.68	4.37	6.32	8.52	11.0	13.6	16.5	19.6	22.8	26.2	29.8
	14	1.08	2.36	3.88	5.62	7.61	9.83	12.3	14.9	17.8	20.8	24.0	27.3
	16	0.96	2.11	3.47	5.05	6.86	8.89	11.1	13.6	16.2	19.0	22.0	25.2
	18	0.87	1.91	3.14	4.57	6.24	8.10	10.2	12.4	14.9	17.5	20.3	23.3
	20	0.79	1.74	2.86	4.18	5.71	7.43	9.35	11.5	13.8	16.2	18.9	21.6
	24	0.67	1.48	2.43	3.56	4.88	6.36	8.03	9.87	11.9	14.1	16.4	18.9
	28	0.58	1.28	2.11	3.10	4.25	5.55	7.02	8.65	10.4	12.4	14.5	16.7
	32	0.51	1.13	1.87	2.74	3.76	4.92	6.23	7.69	9.29	11.0	12.9	14.9
	36	0.46	1.01	1.67	2.45	3.37	4.41	5.60	6.91	8.36	9.95	11.7	13.5
6	2	3.14	6.75	10.7	14.7	18.7	22.7	26.7	30.7	34.7	38.6	42.6	46.6
	3	2.79	6.12	9.94	13.9	18.0	22.0	26.1	30.1	34.1	38.1	42.1	46.1
	4	2.50	5.58	9.23	13.1	17.2	21.2	25.3	29.4	33.4	37.5	41.5	45.5
	5	2.25	5.13	8.58	12.4	16.3	20.4	24.5	28.6	32.7	36.7	40.8	44.8
	6	2.04	4.73	8.00	11.6	15.5	19.5	23.6	27.7	31.8	35.9	40.0	44.1
	7	1.85	4.38	7.47	10.9	14.7	18.7	22.7	26.8	31.0	35.1	39.2	43.3
	8	1.69	4.06	6.98	10.3	14.0	17.9	21.9	25.9	30.1	34.2	38.3	42.4
	9	1.55	3.78	6.55	9.72	13.3	17.1	21.0	25.1	29.2	33.3	37.4	41.5
	10	1.43	3.53	6.15	9.18	12.6	16.3	20.2	24.2	28.3	32.4	36.5	40.6
	12	1.23	3.10	5.47	8.25	11.4	14.9	18.6	22.5	26.5	30.6	34.7	38.8
	14	1.08	2.76	4.90	7.46	10.4	13.7	17.2	21.0	24.9	28.8	32.9	37.0
	16	0.96	2.48	4.43	6.79	9.55	12.6	16.0	19.6	23.3	27.2	31.2	35.2
	18	0.87	2.25	4.04	6.22	8.79	11.7	14.9	18.3	21.9	25.7	29.5	33.5
	20	0.79	2.06	3.70	5.72	8.14	10.9	13.9	17.1	20.6	24.2	28.0	31.9
	24	0.67	1.76	3.17	4.93	7.06	9.48	12.2	15.2	18.3	21.7	25.3	28.9
	28	0.58	1.53	2.76	4.32	6.22	8.38	10.8	13.5	16.5	19.6	22.9	26.3
	32	0.51	1.35	2.45	3.84	5.54	7.50	9.73	12.2	14.9	17.8	20.9	24.1
	36	0.46	1.21	2.19	3.46	5.00	6.77	8.82	11.1	13.6	16.3	19.1	22.2

Table 7–14 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 45°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

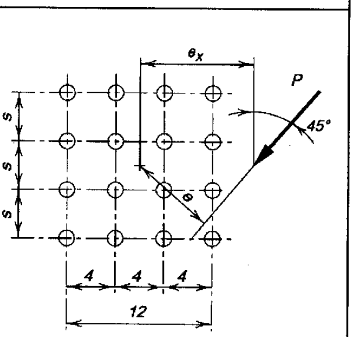
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.46	6.96	10.5	14.2	18.0	21.8	25.7	29.6	33.5	37.4	41.4	45.3
	3	3.15	6.38	9.73	13.2	16.8	20.6	24.4	28.2	32.1	36.1	40.0	44.0
	4	2.87	5.84	8.97	12.3	15.7	19.3	23.1	26.9	30.7	34.6	38.6	42.5
	5	2.61	5.36	8.30	11.4	14.7	18.2	21.8	25.5	29.3	33.2	37.1	41.0
	6	2.39	4.93	7.69	10.7	13.9	17.2	20.7	24.3	28.0	31.8	35.6	39.5
	7	2.19	4.55	7.15	9.98	13.0	16.2	19.6	23.1	26.7	30.4	34.2	38.1
	8	2.01	4.21	6.66	9.34	12.2	15.3	18.6	22.0	25.5	29.2	32.9	36.7
	9	1.86	3.90	6.21	8.76	11.5	14.5	17.7	21.0	24.4	27.9	31.6	35.3
	10	1.72	3.63	5.82	8.24	10.9	13.8	16.8	20.0	23.3	26.8	30.4	34.0
	12	1.49	3.18	5.14	7.33	9.76	12.4	15.2	18.3	21.4	24.7	28.1	31.6
	14	1.32	2.82	4.59	6.58	8.81	11.3	13.9	16.7	19.7	22.8	26.1	29.5
	16	1.17	2.53	4.14	5.95	8.00	10.3	12.7	15.4	18.2	21.2	24.3	27.5
	18	1.06	2.29	3.76	5.43	7.32	9.44	11.7	14.2	16.9	19.7	22.7	25.7
	20	0.96	2.10	3.44	4.98	6.74	8.71	10.9	13.2	15.7	18.4	21.2	24.2
	24	0.82	1.79	2.94	4.26	5.81	7.53	9.43	11.5	13.8	16.2	18.7	21.4
	28	0.71	1.56	2.56	3.73	5.09	6.61	8.31	10.2	12.2	14.4	16.7	19.2
	32	0.63	1.38	2.26	3.31	4.52	5.89	7.42	9.11	11.0	12.9	15.1	17.3
	36	0.56	1.23	2.03	2.97	4.06	5.30	6.69	8.23	9.91	11.7	13.7	15.8
6	2	3.46	7.09	10.9	14.8	18.7	22.7	26.7	30.6	34.6	38.5	42.5	46.5
	3	3.15	6.58	10.3	14.1	18.1	22.0	26.0	30.0	33.9	37.9	41.9	45.9
	4	2.87	6.09	9.65	13.4	17.3	21.3	25.3	29.3	33.3	37.3	41.2	45.2
	5	2.61	5.66	9.07	12.8	16.6	20.6	24.5	28.5	32.5	36.5	40.5	44.5
	6	2.39	5.26	8.54	12.1	15.9	19.8	23.8	27.8	31.8	35.8	39.8	43.8
	7	2.19	4.91	8.07	11.6	15.3	19.1	23.0	27.0	31.0	35.0	39.0	43.0
	8	2.01	4.59	7.63	11.0	14.6	18.4	22.3	26.2	30.2	34.2	38.2	42.2
	9	1.86	4.30	7.23	10.5	14.0	17.7	21.5	25.5	29.4	33.4	37.4	41.4
	10	1.72	4.04	6.85	10.0	13.4	17.1	20.8	24.7	28.6	32.6	36.6	40.6
	12	1.49	3.59	6.19	9.14	12.4	15.9	19.5	23.3	27.2	31.1	35.1	39.1
	14	1.32	3.22	5.62	8.38	11.4	14.8	18.3	22.0	25.8	29.6	33.5	37.5
	16	1.17	2.91	5.13	7.71	10.6	13.8	17.2	20.8	24.4	28.2	32.1	36.0
	18	1.06	2.66	4.71	7.12	9.87	12.9	16.2	19.6	23.2	26.9	30.7	34.6
	20	0.96	2.44	4.35	6.61	9.22	12.1	15.3	18.6	22.1	25.7	29.4	33.2
	24	0.82	2.10	3.76	5.76	8.11	10.8	13.7	16.7	20.0	23.4	27.0	30.6
	28	0.71	1.83	3.30	5.08	7.22	9.64	12.3	15.2	18.3	21.5	24.9	28.4
	32	0.63	1.63	2.94	4.54	6.50	8.71	11.2	13.9	16.7	19.8	23.0	26.3
	36	0.56	1.46	2.64	4.11	5.90	7.93	10.2	12.7	15.4	18.3	21.3	24.5

Table 7–14 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 60°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

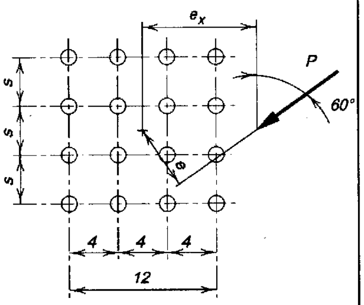
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.74	7.46	11.2	14.9	18.6	22.4	26.2	30.0	33.9	37.7	41.6	45.5
	3	3.57	7.12	10.7	14.3	17.9	21.6	25.3	29.0	32.8	36.7	40.5	44.4
	4	3.38	6.75	10.2	13.6	17.1	20.7	24.3	28.0	31.8	35.6	39.4	43.2
	5	3.17	6.36	9.61	12.9	16.4	19.8	23.4	27.0	30.7	34.5	38.2	42.0
	6	2.97	5.99	9.09	12.3	15.6	19.0	22.5	26.1	29.7	33.4	37.1	40.9
	7	2.78	5.63	8.59	11.7	14.9	18.2	21.6	25.1	28.7	32.3	36.0	39.8
	8	2.60	5.29	8.13	11.1	14.2	17.5	20.8	24.3	27.8	31.4	35.0	38.7
	9	2.44	4.98	7.69	10.6	13.6	16.8	20.1	23.4	26.9	30.4	34.0	37.7
	10	2.28	4.69	7.28	10.1	13.0	16.1	19.3	22.7	26.1	29.5	33.1	36.7
	12	2.02	4.18	6.56	9.16	11.9	14.9	18.0	21.2	24.5	27.8	31.3	34.8
	14	1.80	3.76	5.95	8.38	11.0	13.8	16.7	19.8	23.0	26.3	29.6	33.1
	16	1.62	3.40	5.43	7.70	10.2	12.8	15.6	18.6	21.6	24.8	28.1	31.4
	18	1.47	3.10	4.99	7.11	9.42	11.9	14.6	17.4	20.4	23.5	26.7	29.9
	20	1.34	2.85	4.61	6.59	8.76	11.1	13.7	16.4	19.3	22.2	25.3	28.5
	24	1.15	2.45	3.99	5.73	7.67	9.82	12.2	14.6	17.3	20.1	23.0	26.0
	28	1.00	2.15	3.51	5.06	6.80	8.76	10.9	13.2	15.6	18.2	20.9	23.8
	32	0.88	1.91	3.13	4.52	6.11	7.89	9.83	11.9	14.2	16.6	19.2	21.8
	36	0.79	1.72	2.81	4.08	5.53	7.16	8.95	10.9	13.0	15.3	17.7	20.2
6	2	3.74	7.47	11.2	15.0	18.9	22.8	26.7	30.6	34.5	38.5	42.4	46.4
	3	3.57	7.16	10.8	14.6	18.4	22.2	26.1	30.0	33.9	37.9	41.8	45.8
	4	3.38	6.82	10.4	14.1	17.8	21.7	25.5	29.4	33.3	37.3	41.2	45.1
	5	3.17	6.47	9.94	13.6	17.3	21.1	24.9	28.8	32.7	36.6	40.5	44.5
	6	2.97	6.14	9.52	13.1	16.7	20.5	24.3	28.2	32.1	36.0	39.9	43.8
	7	2.78	5.82	9.11	12.6	16.2	19.9	23.7	27.6	31.5	35.3	39.3	43.2
	8	2.60	5.52	8.73	12.1	15.7	19.4	23.2	27.0	30.8	34.7	38.6	42.5
	9	2.44	5.24	8.37	11.7	15.2	18.9	22.6	26.4	30.2	34.1	38.0	41.9
	10	2.28	4.98	8.03	11.3	14.8	18.4	22.1	25.8	29.7	33.5	37.4	41.3
	12	2.02	4.51	7.41	10.6	14.0	17.5	21.1	24.8	28.5	32.3	36.2	40.1
	14	1.80	4.10	6.86	9.91	13.2	16.6	20.1	23.8	27.5	31.2	35.0	38.9
	16	1.62	3.76	6.37	9.29	12.4	15.8	19.2	22.8	26.5	30.2	33.9	37.7
	18	1.47	3.46	5.94	8.74	11.8	15.0	18.4	21.9	25.5	29.2	32.9	36.6
	20	1.34	3.21	5.56	8.23	11.2	14.3	17.6	21.0	24.6	28.2	31.9	35.6
	24	1.15	2.79	4.91	7.34	10.1	13.0	16.2	19.5	22.9	26.4	30.0	33.6
	28	1.00	2.47	4.38	6.61	9.13	11.9	14.9	18.1	21.4	24.7	28.2	31.8
	32	0.88	2.21	3.95	5.99	8.33	11.0	13.8	16.8	20.0	23.2	26.6	30.1
	36	0.79	2.00	3.58	5.46	7.65	10.1	12.8	15.7	18.7	21.9	25.1	28.5

Table 7-14 (continued)
Coefficients *C* for Eccentrically Loaded Bolt Groups
Angle = 75°

Available Strength of a bolt group, ϕR_n or R_n/Ω, is determined with

$$R_n = C \times r_n$$

$\phi = 0.75$ $\Omega = 2.00$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi r_n}$	$C_{min} = \dfrac{\Omega P_a}{r_n}$

where

P = required force, P_u or P_a, kips

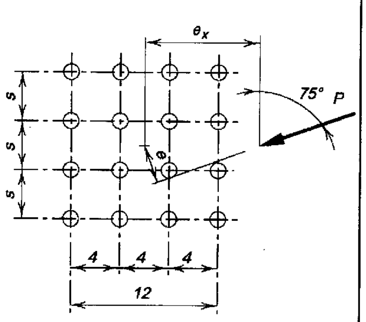
r_n = nominal strength per bolt, kips

e = eccentricity of P with respect to centroid of bolt group, in. (not tabulated, may be determined by geometry)

e_x = horizontal component of e, in.

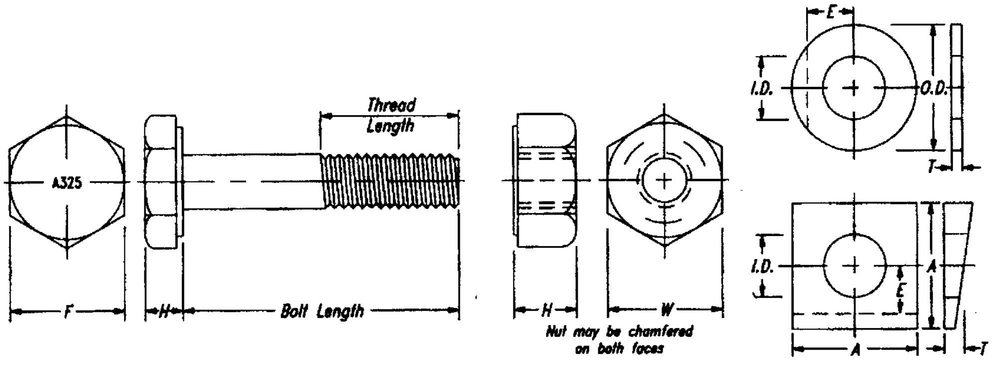
s = bolt spacing, in.

C = coefficient tabulated below



s, in.	e_x, in.	Number of Bolts in One Vertical Row, *n*											
		1	2	3	4	5	6	7	8	9	10	11	12
3	2	3.89	7.75	11.6	15.5	19.3	23.1	26.9	30.8	34.6	38.5	42.3	46.2
	3	3.84	7.66	11.5	15.2	19.0	22.7	26.5	30.3	34.1	37.9	41.7	45.5
	4	3.79	7.54	11.3	15.0	18.7	22.4	26.1	29.8	33.5	37.3	41.0	44.8
	5	3.72	7.40	11.1	14.7	18.3	21.9	25.6	29.3	32.9	36.7	40.4	44.1
	6	3.65	7.25	10.8	14.4	17.9	21.5	25.1	28.7	32.4	36.1	39.8	43.5
	7	3.56	7.08	10.6	14.1	17.6	21.1	24.6	28.2	31.8	35.5	39.1	42.8
	8	3.47	6.90	10.3	13.7	17.2	20.6	24.1	27.7	31.3	34.9	38.5	42.2
	9	3.37	6.71	10.0	13.4	16.8	20.2	23.7	27.2	30.7	34.3	37.9	41.6
	10	3.27	6.52	9.77	13.1	16.4	19.8	23.2	26.7	30.2	33.7	37.3	41.0
	12	3.07	6.14	9.23	12.4	15.6	18.9	22.3	25.7	29.1	32.6	36.2	39.8
	14	2.87	5.76	8.71	11.8	14.9	18.1	21.4	24.7	28.1	31.6	35.1	38.7
	16	2.68	5.40	8.22	11.1	14.2	17.3	20.5	23.8	27.2	30.6	34.1	37.6
	18	2.50	5.07	7.76	10.6	13.5	16.6	19.7	23.0	26.3	29.7	33.1	36.6
	20	2.34	4.76	7.33	10.0	12.9	15.9	19.0	22.2	25.5	28.8	32.2	35.6
	24	2.06	4.23	6.57	9.10	11.8	14.7	17.6	20.7	23.9	27.1	30.4	33.8
	28	1.82	3.78	5.94	8.30	10.9	13.5	16.4	19.3	22.4	25.5	28.7	32.0
	32	1.63	3.41	5.41	7.61	10.0	12.6	15.3	18.1	21.0	24.1	27.2	30.4
	36	1.48	3.11	4.95	7.01	9.26	11.7	14.3	17.0	19.8	22.8	25.8	28.9
6	2	3.89	7.74	11.6	15.4	19.3	23.1	27.0	30.9	35.2	39.1	43.0	47.0
	3	3.84	7.64	11.4	15.2	19.0	22.8	26.6	30.5	34.4	38.3	42.2	46.1
	4	3.79	7.52	11.2	14.9	18.7	22.5	26.3	30.1	34.0	37.8	41.7	45.6
	5	3.72	7.38	11.0	14.7	18.4	22.1	25.9	29.7	33.6	37.4	41.3	45.2
	6	3.65	7.23	10.8	14.4	18.1	21.8	25.6	29.3	33.2	37.0	40.8	44.7
	7	3.56	7.07	10.6	14.2	17.8	21.5	25.2	29.0	32.8	36.6	40.4	44.3
	8	3.47	6.90	10.4	13.9	17.5	21.2	24.9	28.6	32.4	36.2	40.0	43.9
	9	3.37	6.73	10.1	13.6	17.2	20.8	24.5	28.3	32.0	35.8	39.6	43.5
	10	3.27	6.56	9.92	13.4	16.9	20.5	24.2	27.9	31.7	35.5	39.3	43.1
	12	3.07	6.21	9.48	12.9	16.4	19.9	23.6	27.3	31.0	34.7	38.5	42.3
	14	2.87	5.88	9.07	12.4	15.9	19.4	23.0	26.6	30.3	34.1	37.8	41.6
	16	2.68	5.57	8.67	11.9	15.4	18.8	22.4	26.0	29.7	33.4	37.1	40.9
	18	2.50	5.27	8.29	11.5	14.9	18.3	21.9	25.5	29.1	32.8	36.5	40.2
	20	2.34	4.99	7.94	11.1	14.4	17.8	21.3	24.9	28.5	32.2	35.8	39.6
	24	2.06	4.50	7.29	10.3	13.6	16.9	20.4	23.9	27.4	31.0	34.7	38.3
	28	1.82	4.08	6.73	9.67	12.8	16.1	19.4	22.9	26.4	30.0	33.6	37.2
	32	1.63	3.73	6.25	9.06	12.1	15.3	18.6	22.0	25.4	29.0	32.5	36.1
	36	1.48	3.43	5.82	8.51	11.4	14.5	17.8	21.1	24.5	28.0	31.5	35.1

Table 7–15
Dimensions of High-Strength Fasteners, in.



Measurement			Nominal Bolt Diameter, in.								
			$\frac{1}{2}$	$\frac{5}{8}$	$\frac{3}{4}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$
A325 and A490 Bolts[a]	**Width Across Flats, *F***		$\frac{7}{8}$	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{7}{16}$	$1\frac{5}{8}$	$1\frac{13}{16}$	2	$2\frac{3}{16}$	$2\frac{3}{8}$
	Height, *H*		$\frac{5}{16}$	$\frac{25}{64}$	$\frac{15}{32}$	$\frac{35}{64}$	$\frac{39}{64}$	$\frac{11}{16}$	$\frac{25}{32}$	$\frac{27}{32}$	$\frac{15}{16}$
	Thread Length		1	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{3}{4}$	2	2	$2\frac{1}{4}$	$2\frac{1}{4}$
	Bolt Length = Grip + Washer Thickness + →		$\frac{11}{16}$	$\frac{7}{8}$	1	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{1}{2}$	$1\frac{5}{8}$	$1\frac{3}{4}$	$1\frac{7}{8}$
A563 Nuts[b]	**Width Across Flats, *W***		$\frac{7}{8}$	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{7}{16}$	$1\frac{5}{8}$	$1\frac{13}{16}$	2	$2\frac{3}{16}$	$2\frac{3}{8}$
	Height, *H*		$\frac{31}{64}$	$\frac{39}{64}$	$\frac{47}{64}$	$\frac{55}{64}$	$\frac{63}{64}$	$1\frac{7}{64}$	$1\frac{7}{32}$	$1\frac{11}{32}$	$1\frac{15}{32}$
F436 Circular Washers[c]	**Nom. Outside Diameter, *OD***		$1\frac{1}{16}$	$1\frac{5}{16}$	$1\frac{15}{32}$	$1\frac{3}{4}$	2	$2\frac{1}{4}$	$2\frac{1}{2}$	$2\frac{3}{4}$	3
	Nom. Inside Diameter, *ID*		$\frac{17}{32}$	$\frac{11}{16}$	$\frac{13}{16}$	$\frac{15}{16}$	$1\frac{1}{8}$	$1\frac{1}{4}$	$1\frac{3}{8}$	$1\frac{1}{2}$	$1\frac{5}{8}$
	Thckns., *T*	**Min.**	0.097	0.122	0.122	0.136	0.136	0.136	0.136	0.136	0.136
		Max.	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177	0.177
	Min. Edge Distance, *E*[d]		$\frac{7}{16}$	$\frac{9}{16}$	$\frac{21}{32}$	$\frac{25}{32}$	$\frac{7}{8}$	1	$1\frac{3}{32}$	$1\frac{7}{32}$	$1\frac{5}{16}$
F436 Square or Rect. Washers[c,e]	**Min. Side Dimension, *A***		$1\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{3}{4}$	$1\frac{3}{4}$	$2\frac{1}{4}$	$2\frac{1}{4}$	$2\frac{1}{4}$	$2\frac{1}{4}$
	Mean Thickness, *T*		$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$	$\frac{5}{16}$
	Taper in Thickness		2:12	2:12	2:12	2:12	2:12	2:12	2:12	2:12	2:12
	Min. Edge Distance, *E*[d]		$\frac{7}{16}$	$\frac{9}{16}$	$\frac{21}{32}$	$\frac{25}{32}$	$\frac{7}{8}$	1	$1\frac{3}{32}$	$1\frac{7}{32}$	$1\frac{5}{16}$

[a] Tolerances as specified in ASTM A325 and A490.

[b] Tolerances as specified in ASTM A563.

[c] ASTM F436 Washer Tolerances, in.:

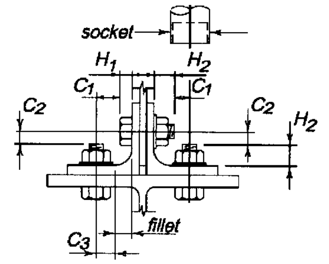
Nominal Outside Diameter	–1/32; +1/32
Nominal Diameter of Hole	–0; +1/32
Flatness: max. deviation from straight-edge placed on cut side shall not exceed	0.010
Concentricity: center of hole to outside diameter (full indicator runout)	0.030
Burr shall not project above immediately adjacent washer surface more than	0.010

[d] For clipped washers only.

[e] For use with American standard beams (S) and channel (C).

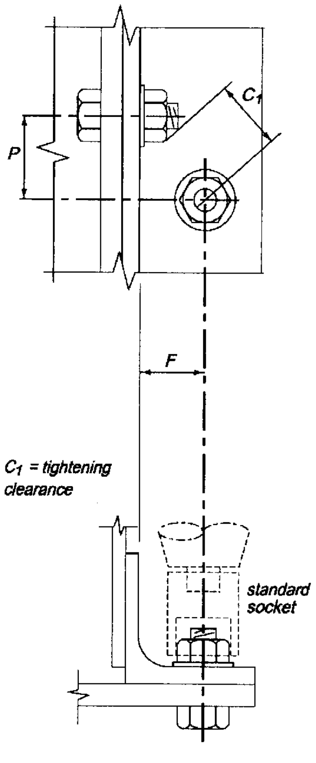
Table 7–16
Entering and Tightening Clearance, in. Conventional ASTM A325 and A490 Bolts

Aligned Bolts



Nominal Bolt Dia.	Socket Dia.	H_1	H_2	C_1	C_2	C_3 Circular	C_3 Clipped
5/8	1 3/4	25/64	1 1/4	1	1 11/16	11/16	9/16
3/4	2 1/4	15/32	1 3/8	1 1/4	3/4	3/4	11/16
7/8	2 1/2	35/64	1 1/2	1 3/8	7/8	7/8	13/16
1	2 5/8	39/64	1 5/8	1 7/16	15/16	1	7/8
1 1/8	2 7/8	11/16	1 7/8	1 9/16	1 1/16	1 1/8	1
1 1/4	3 1/8	25/32	2	1 11/16	1 1/8	1 1/4	1 1/8
1 3/8	3 1/4	27/32	2 1/8	1 3/4	1 1/4	1 3/8	1 1/4
1 1/2	3 1/2	15/16	2 1/4	1 7/8	1 5/16	1 1/2	1 5/16

Staggered Bolts



F	Stagger P, in. Nominal Bolt Diameter, in. 5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
1	1 5/8							
1 1/8	1 1/2							
1 1/4	1 1/2	1 15/16						
1 3/8	1 7/16	1 7/8	2 3/16					
1 1/2	1 1/4	1 13/16	2 1/8	2 5/16				
1 5/8	1 1/4	1 3/4	2 1/16	2 5/16	2 9/16			
1 3/4	1 3/16	1 11/16	2	2 1/4	2 9/16	2 13/16	3	
1 7/8	1 1/8	1 9/16	1 15/16	2 3/16	2 1/2	2 3/4	3	3 3/4
2	1	1 1/2	1 13/16	2 1/8	2 7/16	2 3/4	2 15/16	3 1/4
2 1/8	13/16	1 3/8	1 11/16	2	2 3/8	2 11/16	2 15/16	3 3/16
2 1/4		1 1/4	1 9/16	1 7/8	2 1/4	2 5/8	2 7/8	3 3/16
2 3/8		1 1/8	1 1/2	1 3/4	2 1/8	2 1/2	2 13/16	3 1/8
2 1/2		7/8	1 3/8	1 5/8	2	2 7/16	2 3/4	3 1/16
2 5/8			1 3/16	1 1/2	1 15/16	2 5/16	2 7/8	3
2 3/4			15/16	1 3/8	1 7/8	2 1/8	2 1/2	2 7/8
2 7/8				1 3/16	1 3/4	2 1/16	2 3/8	2 13/16
3				7/8	1 5/8	2	2 1/4	2 11/16
3 1/8					1 1/2	1 7/8	2 1/8	2 1/2
3 1/4					1 1/4	1 3/4	2	2 3/8
3 3/8					15/16	1 5/8	1 15/16	2 1/4
3 1/2						1 3/8	1 3/4	2 1/8
3 5/8						1 1/16	1 9/16	2
3 3/4							1 5/16	1 7/8
3 7/8								1 11/16
4								1 3/8

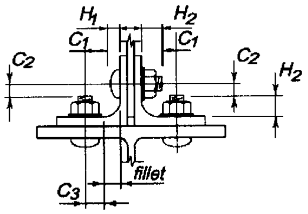
Notes:

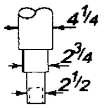
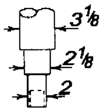
H_1 = height of head
H_2 = maximum shank extension*
C_1 = clearance for tightening
C_2 = clearance for entering
C_3 = clearance for fillet*
P = bolt stagger
F = clearance for tightening staggered bolts
* Based on the use of one ASTM F436 washer.

Table 7–17

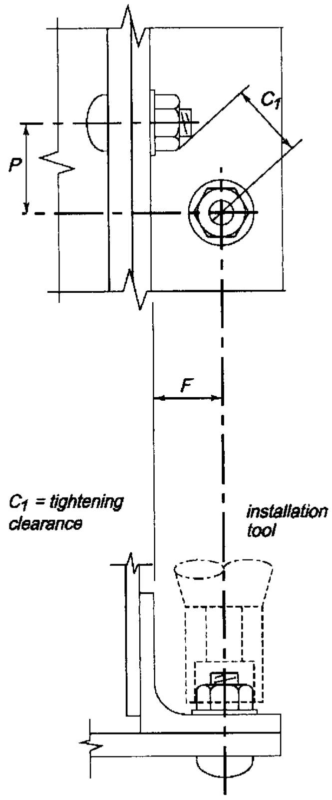
Entering and Tightening Clearance, in. Tension Control ASTM F1852 and A490 Bolts

Aligned Bolts



Tools	Nominal Bolt Dia.	H_1	H_2	C_1	C_2	C_3 Circular	C_3 Clipped
Large Tools 	**4 1/4-in. Diameter Critical**						
	3/4	1/2	1 3/8	1 7/8	7/8	3/4	–
	7/8	9/16	1 1/2	1 7/8	1	7/8	–
	1	5/8	1 3/4	1 7/8	1 1/8	1	–
	2 3/4-in. Diameter Critical						
	3/4	1/2	1 3/8	1 3/8	7/8	3/4	–
	7/8	9/16	1 1/2	1 3/8	1	7/8	–
	1	5/8	1 3/4	1 3/8	1 1/8	1	–
Small Tools 	**3 1/8-in. Diameter Critical**						
	5/8	7/16	1 1/4	1 5/8	13/16	11/16	–
	3/4	1/2	1 3/8	1 5/8	7/8	3/4	–
	7/8	9/16	1 1/2	1 5/8	1	7/8	–
	2 1/8-in. Diameter Critical						
	5/8	7/16	1 1/4	1 1/4	13/16	11/16	–
	3/4	1/2	1 3/8	1 1/4	7/8	3/4	–
	7/8	9/16	1 1/2	1 1/4	1	7/8	–

Staggered Bolts



F	Stagger P, in. — Nominal Bolt Diameter, in. 5/8	3/4	7/8	1
1 1/4	1 13/16			
1 3/8	1 3/4	2 1/16		2 7/16
1 1/2	1 11/16	2	2 1/4	2 3/8
1 5/8	1 9/16	1 7/8	2 3/16	2 1/4
1 3/4	1 1/2	1 13/16	2 1/16	2 3/16
1 7/8	1 7/16	1 3/4	2	2 1/8
2	1 5/16	1 5/8	1 7/8	2
2 1/8	1 1/4	1 9/16	1 3/4	1 15/16
2 1/4	1 3/16	1 1/2	1 11/16	1 7/8
2 3/8	1 1/8	1 3/8	1 9/16	1 3/4
2 1/2	1	1 5/16	1 1/2	1 11/16
2 5/8		1 3/16	1 3/8	1 9/16
2 3/4		1 1/8	1 5/16	1 1/2
2 7/8		1 1/2	1 3/16	1 3/8
3		1 5/16	1 1/8	1 7/8
3 3/8				1 7/16

Notes:

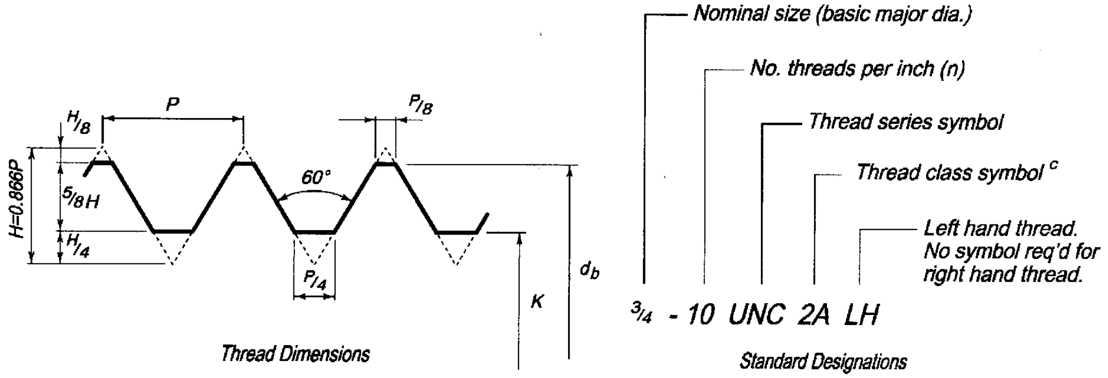
H_1 = height of head
H_2 = maximum shank extension*
C_1 = clearance for tightening
C_2 = clearance for entering
C_3 = clearance for fillet*
P = bolt stagger
F = clearance for tightening staggered bolts
* Based on one standard hardened washer.

Table 7–18
Threading Dimensions for High-Strength and Non-High-Strength Bolts

SCREW THREADS

Unified Standard Series-UNC/UNRC and 4UN/4UNR

ANSI B1.1



Thread Dimensions

Standard Designations

Diameter		Area			
Bolt Diameter d_b, in.	**Min. Root K, in.**	**Gross Bolt Area, in.^{2}**	**Min. Root Area, in.^{2}**	**Net Tensile Area[a], in.^{2}**	**Threads per inch, n[b]**
1/4	0.196	0.0490	0.0301	0.0320	20
3/8	0.307	0.110	0.0742	0.0780	16
1/2	0.417	0.196	0.136	0.142	13
5/8	0.527	0.307	0.218	0.226	11
3/4	0.642	0.442	0.323	0.334	10
7/8	0.755	0.601	0.447	0.462	9
1	0.865	0.785	0.587	0.606	8
1 1/8	0.970	0.994	0.740	0.763	7
1 1/4	1.10	1.23	0.942	0.969	7
1 3/8	1.19	1.49	1.12	1.16	6
1 1/2	1.32	1.77	1.37	1.41	6
1 3/4	1.53	2.41	1.85	1.90	5
2	1.76	3.14	2.43	2.50	4.5
2 1/4	2.01	3.98	3.17	3.25	4.5
2 1/2	2.23	4.91	3.90	4.00	4
2 3/4	2.48	5.94	4.83	4.93	4
3	2.73	7.07	5.85	5.97	4
3 1/4	2.98	8.30	6.97	7.10	4
3 1/2	3.23	9.62	8.19	8.33	4
3 3/4	3.48	11.0	9.51	9.66	4
4	3.73	12.6	10.9	11.1	4

[a] Net tensile area = $0.7854 \times \left(d_b - \dfrac{0.9743}{n}\right)^2$

[b] For diameters listed, thread series is UNC (coarse). For larger diameters, thread series is 4UN.

[c] 2A denotes Class 2A fit applicable to external threads;
2B denotes corresponding Class 2B fit for internal threads.

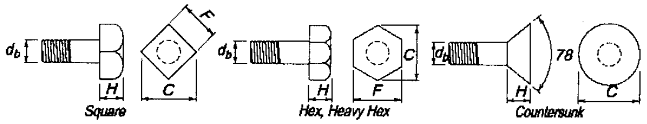
Table 7–19
Weights of High-Strength Fasteners, pounds per 100 count

Bolt Length, in.		Nominal Bolt Diameter, in.								
		1/2	5/8	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
100, Conventional A325 or A490 Bolts with A563 Nuts	1	16.5	29.4	47.0	—	—	—	—	—	—
	1 1/4	17.8	31.1	49.6	74.4	104	—	—	—	—
	1 1/2	19.2	33.1	52.2	78.0	109	148	197	—	—
	1 3/4	20.5	35.3	55.3	81.9	114	154	205	261	333
	2	21.9	37.4	58.4	86.1	119	160	212	270	344
	2 1/4	23.3	39.8	61.6	90.3	124	167	220	279	355
	2 1/2	24.7	41.7	64.7	94.6	130	174	229	290	366
	2 3/4	26.1	43.9	67.8	98.8	135	181	237	300	379
	3	27.4	46.1	70.9	103	141	188	246	310	391
	3 1/4	28.8	48.2	74.0	107	146	195	255	321	403
	3 1/2	30.2	50.4	77.1	111	151	202	263	332	416
	3 3/4	31.6	52.5	80.2	116	157	209	272	342	428
	4	33.0	54.7	83.3	120	162	216	280	353	441
	4 1/4	34.3	56.9	86.4	124	168	223	289	363	453
	4 1/2	35.7	59.0	89.5	128	173	230	298	374	465
	4 3/4	37.1	61.2	92.7	133	179	237	306	384	478
	5	38.5	63.3	95.8	137	184	244	315	395	490
	5 1/4	39.9	65.5	98.9	141	190	251	324	405	503
	5 1/2	41.2	67.7	102	146	196	258	332	416	515
	5 3/4	42.6	69.8	105	150	201	265	341	426	527
	6	44.0	71.9	108	154	207	272	349	437	540
	6 1/4	—	74.1	111	158	212	279	358	447	552
	6 1/2	—	76.3	114	163	218	286	367	458	565
	6 3/4	—	78.5	118	167	223	293	375	468	577
	7	—	80.6	121	171	229	300	384	479	589
	7 1/4	—	82.8	124	175	234	307	392	489	602
	7 1/2	—	84.9	127	179	240	314	401	500	614
	7 3/4	—	87.1	130	183	246	321	410	510	626
	8	—	89.2	133	187	251	328	418	521	639
	8 1/4	—	—	—	192	257	335	427	531	651
	8 1/2	—	—	—	196	262	342	435	542	664
	8 3/4	—	—	—	—	—	—	444	552	676
	9	—	—	—	—	—	—	453	563	689
	Per inch add'tl. Add	5.50	8.60	12.4	16.9	22.1	28.0	34.4	42.5	49.7
100, F436 Circular Washers		2.10	3.60	4.80	7.00	9.40	11.3	13.8	16.8	20.0
100, F436 Square Washers		23.1	22.4	21.0	20.2	19.2	34.0	31.6	31.2	32.9

This table conforms to weight standards adopted by the Industrial Fasteners Institute (IFI), updated for washer weights.

Table 7–20

Dimensions of Non-High-Strength Bolts, in.



	Bolts Dia d_b, in.	Square			Hex			Heavy Hex			Countersunk		Min, Thrd. Length, in.	
		F, in.	*C*, in.	*H*, in.	*F*, in.	*C*, in.	*H*, in.	*F*, in.	*C*, in.	*H*, in.	*C*, in.	*H*, in.	*L* ≤ 6 in.	*L* > 6 in.
Bolts	**1/4**	3/8	1/2	3/16	7/16	1/2	3/16	–	–	–	1/2	1/8	3/4	1
	3/8	9/16	13/16	1/4	9/16	5/8	1/4	–	–	–	11/16	3/16	1	1 1/4
	1/2	3/4	1 1/16	5/16	3/4	7/8	3/8	7/8	1	3/8	7/8	1/4	1 1/4	1 1/2
	5/8	15/16	1 5/16	7/16	15/16	1 1/16	7/16	1 1/16	1 1/4	1 1/8	1 1/8	5/16	1 1/2	1 3/4
	3/4	1 1/8	1 9/16	1/2	1 1/8	1 5/16	1/2	1 1/4	1 7/16	1/2	1 3/8	3/8	1 3/4	2
	7/8	1 5/16	1 7/8	5/8	1 5/16	1 1/2	9/16	1 7/16	1 11/16	9/16	1 9/16	7/16	2	2 1/4
	1	1 1/2	2 1/8	11/16	1 1/2	1 3/4	11/16	1 5/8	1 7/8	11/16	1 13/16	1/2	2 1/4	2 1/2
	1 1/8	1 11/16	2 3/8	3/4	1 11/16	1 15/16	3/4	1 13/16	2 1/16	3/4	2 1/16	9/16	2 1/2	2 3/4
	1 1/4	1 7/8	2 5/8	7/8	1 7/8	2 3/16	7/8	2	2 5/16	7/8	2 1/4	5/8	2 3/4	3
	1 3/8	2 1/16	2 15/16	15/16	2 1/16	2 3/8	15/16	2 3/16	2 1/2	15/16	2 1/2	11/16	3	3 1/4
	1 1/2	2 1/4	3 3/16	1	2 1/4	2 5/8	1	2 3/8	2 3/4	1	2 11/16	3/4	3 1/4	3 1/2
	1 3/4	–	–	–	2 5/8	3	1 3/16	2 3/4	3 3/16	1 3/16	–	–	3 3/4	4
	2	–	–	–	3	3 7/16	1 3/8	3 1/8	3 5/8	1 3/8	–	–	4 1/4	4 1/2
	2 1/4	–	–	–	3 3/8	3 7/8	1 1/2	3 1/2	4 1/16	1 1/2	–	–	4 3/4	5
	2 1/2	–	–	–	3 3/4	4 5/16	1 11/16	3 7/8	4 1/2	1 11/16	–	–	5 1/4	5 1/2
	2 3/4	–	–	–	4 1/8	4 3/4	1 13/16	4 1/4	4 15/16	1 13/16	–	–	5 3/4	6
	3	–	–	–	4 1/2	5 3/16	2	4 5/8	5 5/16	2	–	–	6	6 1/2
	3 1/4	–	–	–	4 7/8	5 5/8	2 3/16	–	–	–	–	–	6	7
	3 1/2	–	–	–	5 1/4	6 1/16	2 5/16	–	–	–	–	–	6	7 1/2
	3 3/4	–	–	–	5 5/8	6 1/2	2 1/2	–	–	–	–	–	6	8
	4	–	–	–	6	6 15/16	2 11/16	–	–	–	–	–	6	8 1/2

Notes:

For high-strength bolt and nut dimensions, refer to Table 7–15.

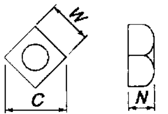
Square, hex, and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ANSI B18.2.1.

Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ANSI 18.5.

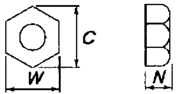
Minimum thread length = $2d_b$ + 1/4-in. for bolts up to 6 in. long, and $2d_b$ + 1/2-in. for bolts longer than 6 in.

Table 7–20 (continued)

Dimensions of Non-High-Strength Nuts, in.



Square, Heavy Square



Hex, Heavy Hex

Nut Size, in.		Square			Hex			Heavy Square			Heavy Hex		
		W, in.	C, in.	N, in.	W, in.	C, in.	N, in.	W, in.	C, in.	N, in.	W, in.	C, in.	N, in.
Nuts	1/4	7/16	5/8	1/4	7/16	1/2	3/16	1/2	11/16	1/4	1/2	9/16	1/4
	3/8	5/8	7/8	5/16	9/16	5/8	1/4	11/16	1	3/8	11/16	13/16	3/8
	1/2	4/5	1 1/8	7/16	3/4	7/8	3/8	7/8	1 1/4	1/2	7/8	1	1/2
	5/8	1	1 7/16	9/16	15/16	1 1/16	7/16	1 1/16	1 1/2	5/8	1 1/16	1 1/4	5/8
	3/4	1 1/8	1 9/16	11/16	1 1/8	1 5/16	1/2	1 1/4	1 3/4	3/4	1 1/4	1 7/16	3/4
	7/8	1 5/16	1 7/8	3/4	1 5/16	1 1/2	9/16	1 7/16	2 1/16	7/8	1 7/16	1 11/16	7/8
	1	1 1/2	2 1/8	7/8	1 1/2	1 3/4	11/16	1 5/8	2 5/16	1	1 5/8	1 7/8	1
	1 1/8	1 11/16	2 3/8	1	1 11/16	1 15/16	3/4	1 13/16	2 9/16	1 1/8	1 13/16	2 1/16	1 1/8
	1 1/4	1 7/8	2 5/8	1 1/8	1 7/8	2 3/16	7/8	2	2 13/16	1 1/4	2	2 5/16	1 1/4
	1 3/8	2 1/16	2 15/16	1 1/4	2 1/16	2 3/8	15/16	2 3/16	3 1/8	1 3/8	2 3/16	2 1/2	1 3/8
	1 1/2	2 1/4	3 3/16	1 5/16	2 1/4	2 5/8	1	2 3/8	3 3/8	1 1/2	2 3/8	2 3/4	1 1/2
	1 3/4	–	–	–	–	–	–	–	–	–	2 3/4	3 3/16	1 3/4
	2	–	–	–	–	–	–	–	–	–	3 1/8	3 5/8	2
	2 1/4	–	–	–	–	–	–	–	–	–	3 1/2	4 1/16	2 3/16
	2 1/2	–	–	–	–	–	–	–	–	–	3 7/8	4 1/2	2 7/16
	2 3/4	–	–	–	–	–	–	–	–	–	4 1/4	4 15/16	2 11/16
	3	–	–	–	–	–	–	–	–	–	4 5/8	5 5/16	2 15/16
	3 1/4	–	–	–	–	–	–	–	–	–	5	5 3/4	3 3/16
	3 1/2	–	–	–	–	–	–	–	–	–	5 3/8	6 3/16	3 7/16
	3 3/4	–	–	–	–	–	–	–	–	–	5 3/4	6 5/8	3 11/16
	4	–	–	–	–	–	–	–	–	–	6 1/8	7 1/16	3 15/16

Notes:

For high-strength bolt and nut dimensions, refer to Table 7–15.

Square, hex, and heavy hex bolt dimensions, rounded to nearest 1/16 in., are in accordance with ANSI B18.2.1.

Countersunk bolt dimensions, rounded to the nearest 1/16 in., are in accordance with ANSI 18.5.

Minimum thread length = 2d_b + 1/4-in. for bolts up to 6 in. long, and 2d_b + 1/2-in. for bolts longer than 6 in.

Table 7-21
Weights of Non-High-Strength Fasteners, pounds

	Bolt Length, in.	Nominal Bolt Diameter, in.								
		1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
100 Square Bolts with Hexagonal Nuts*	1	2.38	6.11	13.0	24.1	38.9	–	–	–	–
	1 1/4	2.71	6.71	14.0	25.8	41.5	–	–	–	–
	1 1/2	3.05	7.47	15.1	27.6	44.0	67.3	95.1	–	–
	1 3/4	3.39	8.23	16.5	29.3	46.5	70.8	99.7	–	–
	2	3.73	8.99	17.8	31.4	49.1	74.4	104	143	–
	2 1/4	4.06	9.75	19.1	33.5	52.1	77.9	109	149	–
	2 1/2	4.40	10.5	20.5	35.6	55.1	82.0	114	155	206
	2 3/4	4.74	11.3	21.8	37.7	58.2	86.1	119	161	213
	3	5.07	12.0	23.2	39.8	61.2	90.2	124	168	221
	3 1/4	5.41	12.8	24.5	41.9	64.2	94.4	129	174	229
	3 1/2	5.75	13.5	25.9	44.0	67.2	98.5	135	181	237
	3 3/4	6.09	14.3	27.2	46.1	70.2	103	140	188	246
	4	6.42	15.1	28.6	48.2	73.3	107	145	195	254
	4 1/4	6.76	15.8	29.9	50.3	76.3	111	151	202	262
	4 1/2	7.10	16.6	31.3	52.3	79.3	115	156	208	271
	4 3/4	7.43	17.3	32.6	54.4	82.3	119	162	215	279
	5	7.77	18.1	33.9	56.5	85.3	123	167	222	288
	5 1/4	8.11	18.9	35.3	58.6	88.4	127	172	229	296
	5 1/2	8.44	19.6	36.6	60.7	91.4	131	178	236	304
	5 3/4	8.78	20.4	38.0	62.8	94.4	136	183	242	313
	6	9.12	21.1	39.3	64.9	97.4	140	188	249	321
	6 1/4	9.37	21.7	40.4	66.7	100	143	193	255	329
	6 1/2	9.71	22.5	41.8	68.7	103	147	198	262	337
	6 3/4	10.1	23.3	43.1	70.8	106	151	204	269	345
	7	10.4	24.0	44.4	72.9	109	156	209	275	354
	7 1/4	10.7	24.8	45.8	75.0	112	160	214	282	362
	7 1/2	11.0	25.5	47.1	77.1	115	164	220	289	371
	7 3/4	11.4	26.3	48.5	79.2	118	168	225	296	379
	8	11.7	27.0	49.8	81.3	121	172	231	303	387
	8 1/2	–	28.6	52.5	85.5	127	180	241	316	404
	9	–	30.1	55.2	89.7	133	189	252	330	421
	9 1/2	–	31.6	57.9	93.9	139	197	263	343	438
	10	–	66.1	60.6	98.1	145	205	274	357	454
	10 1/2	–	34.6	63.3	102	151	213	284	371	471
	11	–	36.2	66.0	106	157	221	295	384	488
	11 1/2	–	37.7	68.7	110	163	230	306	398	505
	12	–	39.2	71.3	115	170	238	316	411	522
	12 1/2	–	–	74.0	119	176	246	327	425	538
	13	–	–	76.7	123	182	254	338	439	556
	13 1/2	–	–	79.4	127	188	263	349	452	572
	14	–	–	82.1	131	194	271	359	466	589
	14 1/2	–	–	84.8	135	200	279	370	479	605
	15	–	–	87.5	140	206	287	381	493	622
	15 1/2	–	–	90.2	144	212	296	392	507	639
	16	–	–	92.9	148	218	304	402	520	656
	Per inch add'tl. Add	1.3	3.0	5.4	8.4	12.1	16.5	21.4	27.2	33.6

Notes:

For weight of high-strength fasteners, see Table 7-20.

This table conforms to weight standards adopted by the Industrial Fasteners Institute (IFI).

*Square bolt per ANSI B 18.2.1, hexagonal nut per ANSI B18.2.2. For other non-high-strength fasteners, refer to Tables 7-22 and 7-23.

Table 7–22
Weight Adjustments
for Combinations of Non-High-Strength Fasteners Other than Tabulated in Table 7–21

Combinations of 100		Add or Subtr.	Nominal Bolt Diameter, in.								
			1/4	3/8	1/2	5/8	3/4	7/8	1	1 1/8	1 1/4
Square Bolts With	**Square Nuts**	+	0.1	1.0	2.0	3.4	3.5	5.5	8.0	12.2	16.3
	Heavy Square Nuts	+	0.6	2.1	4.1	7.0	11.6	17.2	23.2	32.1	41.2
	Heavy Hex Nuts	+	0.4	1.5	2.8	4.6	7.6	10.7	14.2	18.9	24.3
100, Square Bolts with Hexagonal Nuts*	**Square Nuts**	+	0.1	0.6	1.1	1.4	0.2	0.5	-0.2	-0.1	-1.7
	Hex Nuts	–	0.0	0.4	0.9	2.0	3.3	5.0	8.2	12.3	18.0
	Heavy Square Nuts	+	0.6	1.7	3.2	5.0	8.3	12.2	15.0	19.8	23.2
	Heavy Hex Nuts	+	0.4	1.1	1.9	2.6	4.3	5.7	6.0	6.6	6.3
100, Hex Bolts	**Heavy Square Nuts**	+	–	–	4.7	7.3	11.3	16.5	20.7	27.0	33.6
	Heavy Hex Nuts	+	–	–	3.4	4.9	7.3	10.0	11.7	13.8	16.7

Notes:
For weights of high-strength fasteners, see Table 7–19.
This table conforms to weight standards adopted by the Industrial Fasteners Institute (IFI).
*Add or subtract value in this table to or from the value in Table 7–21.

Table 7–23

Weights of Non-High-Strength Bolts

of Diameter Greater Than 1¼ in., pounds

Weight of 100 Each		Nominal Bolt Diameter, in.											
		$1^{3}/_{8}$	$1^{1}/_{2}$	$1^{3}/_{4}$	2	$2^{1}/_{4}$	$2^{1}/_{2}$	$2^{3}/_{4}$	3	$3^{1}/_{4}$	$3^{1}/_{2}$	$3^{3}/_{4}$	4
Heads of:	**Square Bolts**	105	130	–	–	–	–	–	–	–	–	–	–
	Hex Bolts	84.0	112	178	259	369	508	680	900	1120	1390	1730	2130
	Heavy Hex Bolts	95.0	124	195	280	397	541	720	950	–	–	–	–
One Linear Inch, Unthreaded Shank		42.0	50.0	68.2	89.0	113	139	168	200	235	272	313	356
One Linear Inch, Threaded Shank		35.0	42.5	57.4	75.5	97.4	120	147	178	210	246	284	325
Square Nuts		94.5	122	–	–	–	–	–	–	–	–	–	–
Heavy Square Nuts		125	161	–	–	–	–	–	–	–	–	–	–
Heavy Hex Nuts		102	131	204	299	419	564	738	950	1190	1530	1810	2180

– Indicates that the bolt size is not available.

PART 8

DESIGN CONSIDERATIONS FOR WELDS

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of welded joints. For the design of connecting elements, see Part 9. For the design of simple shear, moment, bracing, and other connections, see Parts 10 through 15. For welded joints that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

GENERAL REQUIREMENTS FOR WELDED JOINTS

The requirements for welded construction are given in AISC Specification Section M2.4, which requires the use of AWS D1.1, except as modified in AISC Specification Section J2. For further information see also Blodgett et al. (1997).

Welding in structural steel in performed in compliance with written welding procedure specifications (WPSs). WPSs are qualified by test or prequalified in AWS D1.1. WPSs are used to control base metal, consumables, joint geometry, electrical, and other essential variables for welded joints.

Consumables

Requirements for welding consumables are given in AISC Specification Sections A3.5, J2.6, and J2.7. Permissible filler metal strengths are shown in Table J2.5, based on matching filler metals shown in AWS D1.1 Table 3.1. Filler metal notch-toughness requirements are given in AISC Specification J2.6. Low-hydrogen electrodes for SMAW are required, as shown in AWS D1.1 Table 3.1. Low-hydrogen SMAW electrodes have a limited exposure time and rod ovens are necessary near the point of use for storage.

Requirements for the manufacture, classification, and packing of consumables are given in AWS A5.x specifications. Consumables vary based upon their welding process. Shielded Metal Arc Welding (SMAW), or 'stick' welding, is a manual process. Submerged Arc Welding (SAW) is a semiautomatic or automatic process. Consumables are classified as an electrode flux combination because the weld metal properties are dependant on both the electrode and the flux. SAW is suitable for long straight or circumferential welds but the work must be performed in horizontal or flat positions. Flux-Cored Arc Welding (FCAW) uses wire electrode that contains flux in the center. FCAW electrodes are provided for use with a gas shield or self shield. Gas for shielding is argon, carbon dioxide, or a combination of the two. Gas Metal Arc Welding (GMAW) uses wire electrodes that are solid or have a metal core. GMAW is performed with gas shielding.

Thermal Cutting

Oxygen-fuel gas cutting can be used to cut almost any commercially available plate thickness. If the plate being cut contains large discontinuities or non-metallic inclusions, turbulence may be created in the cutting stream, resulting in notches or gouges in the edge of the cut. Plasma-arc cutting is much faster and less susceptible to the effects of discontinuities or non-metallic inclusions, but leaves a slight taper in the cut as it descends and can be used only up to about $1^1/2$ in. thickness. Within the depth limits given in the AISC Specification and AWS D1.1, it is usually better practice to remove and fair-in notches or gouges by grinding than to weld repair and grind. In either case, however, a smooth transition should be provided.

Air-Arc Gouging

In this method, a carbon arc is used to melt a nugget-shaped area of the base metal, which is blown away with a jet of compressed air. Air-arc gouging can be used to remove weld defects, gouge the weld root to sound weld metal, form a U groove on one side of a square butt joint, and for similar operations.

Inspection

The five most commonly used methods for welding inspection are discussed below and in AWS Guide for the Non-Destructive Inspection of Welds (B1.0). The designer must specify in the contract documents the types of weld inspection required as well as the extent and application of each type of inspection. In the absence of instructions for weld inspection, the fabricator or erector is only responsible for those weld discontinuities found by visual inspection (see AWS D1.1). If additional inspection more stringent than visual is later required, the owner is normally responsible for the costs of inspection and of weld repairs other than those identified by the visual inspection requirements. Weld repairs which may be difficult to perform and which may potentially damage other aspects of the connection are best referred to the engineer of record to determine the necessity of the correction with due consideration of fitness for purpose.

Visual inspection is the most commonly required inspection process. The designer must realize that more stringent requirements for inspection can needlessly add significant cost to the project and should specify them only in those instances where they are essential to the integrity of the structure.

Visual Testing (VT)

Visual inspection provides the most economical way to check weld quality and is the most commonly used method. Joints are scrutinized prior to the commencement of welding to check fit-up, preparation bevels, gaps, alignment, and other variables. After the joint is welded, it is then visually inspected in accordance with AWS D1.1. If a discontinuity is suspected, the weld is either repaired or other inspection methods are used to validate the integrity of the weld. In most cases, timely visual inspection by an experienced inspector is sufficient and offers the most practical and effective inspection alternative to other, more costly methods.

Penetrant Testing (PT)

This test uses a red dye penetrant applied to the work from a pressure spray can. The dye penetrates any crack or crevice open to the surface. Excess dye is removed and white developer is sprayed on. Dye seeps out of the crack, producing a red image on the white developer (See Figure 8-1).

PT can be used to detect tight cracks as long as they are open to the surface. However, only surface cracks are detectable. Furthermore, deep weld ripples and scratches may give a false indication when PT is used.

Dye penetrant examination tends to be messy and slow, but can be helpful when determining the extent of a defect found by visual inspection. This is especially true when a defect is being removed by gouging or grinding for the repair of a weld to assure that the defect is completely removed.

Magnetic-Particle Testing (MT)

A magnetizing current is introduced with a yoke or contact prods into the weldment to be inspected, as sketched in Figure 8-2 (prods shown). This induces a magnetic field in the work, which will be distorted by any cracks, seams, inclusions, etc. located on or near (within approx. 0.1 in. of) the surface. A dry magnetic powder, blown lightly on the surface by a rubber squirt bulb, will be picked up at such discontinuities, making a distinct mark. The magnetically held particles show the location, size, and shape of the discontinuity.

The method will indicate surface cracks that might be difficult for liquid penetrant to enter and subsurface cracks to about 0.1-in depth, with proper magnetization. Records may be kept by picking up the powder pattern with clear plastic tape. Cleanup is easy, but demag-

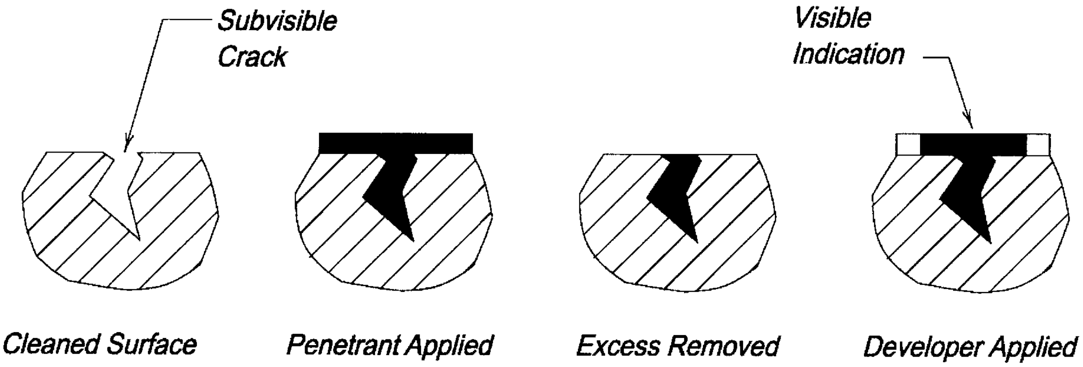


Figure 8–1. Schematic illustration of penetrant testing (PT).

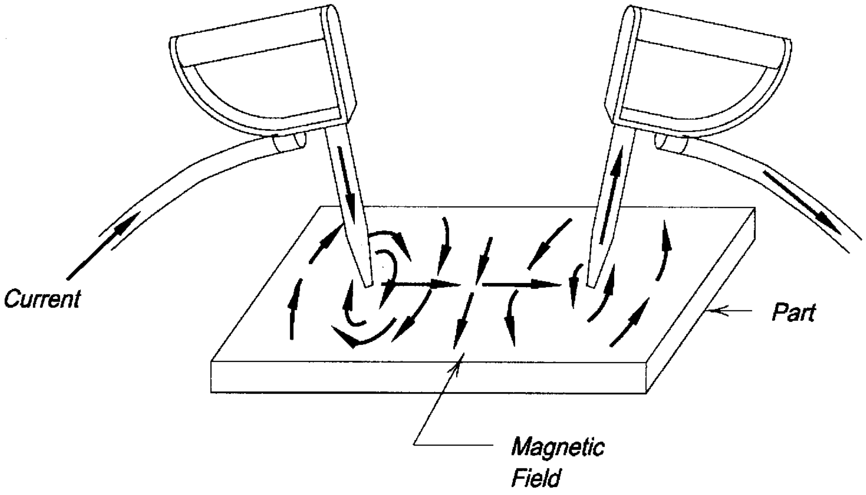


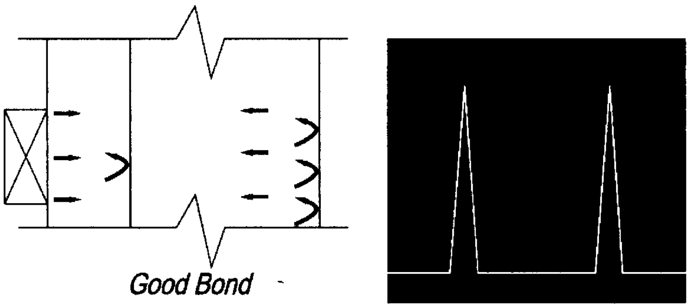
Figure 8–2. Schematic illustration of magnetic particle testing (MT).

netizing, if necessary, may not be. If the magnetizing prod is lifted from the work while the current is still on, an arc strike which could lead to cracking could result. If arc strikes occur, they should be ground out.

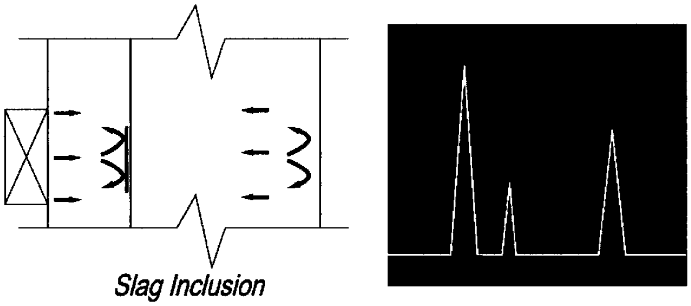
Magnetic particle examination can be useful when a defect is suspected from visual inspection or when the absence of cracking in areas of high restraint must be confirmed. Relatively smooth surfaces are required for MT and, while cleanup is easy, demagnetization, when necessary, may not be.

Ultrasonic Testing (UT)

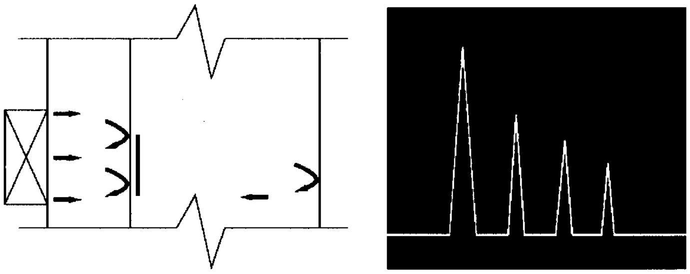
The ultrasonic (UT) inspection process is analogous to radar. A short pulse of high-frequency sound is broadcast from a crystal into a metal, after which the crystal waits to receive reflections from the far end of the metal member and from any voids encountered on the way through. The technique is called pulse echo. The sound beam is produced by a piezoelectric transducer energized by an electric current which causes the crystal to vibrate and transmit through a liquid couplant into the metal. Any reflections are displayed as pips on a cathode ray tube (CRT) grid whose horizontal scale represents distance through the metal. The vertical scale represents the strength (or area) of the reflecting surface. The system is shown schematically in Figure 8-3.



Good Bond



Slag Inclusion



Crack or
Incomplete Fusion

Figure 8–3. Variations in UT reflections caused by defects at the boundary.

The accuracy of ultrasonic inspection is highly dependent upon the skill and training of the operator and frequent calibration of the instrument. There is a "dead" area beneath most transducers that makes it difficult to inspect members less than $^5/_{16}$ in. in thickness. Austenitic stainless steels and extremely coarse-grained steels, e.g., electroslag welds, are difficult to inspect; but on structural carbon and low-alloy steels, the process can detect flat discontinuities (favorably oriented for reflection) smaller than $^1/_{64}$ in. The crystal, which is $^3/_8$ in. to 1 in. in size, can be readily moved about to check many orientations and can project the beam into the metal at angles of 90°, 70°, 60°, and 45°. With the latter three angles, the beam can be bounced around inside the metal, producing echoes from any discontinuity on the way. For more information see Krautkramer (1977) and Institute of Welding (1972).

Ultrasonic testing is a more versatile, rapid, and economical inspection method than radiography, but it does not provide a permanent record like the X-ray negative. The operator, instead, makes a written record of discontinuity indications appearing on his cathode ray tube (CRT). Certain joint geometry limits the use of the ultrasonic method.

Ultrasonic examination has limited applicability in some applications, such as HSS fabrication. Relatively thin sections and variations in joint geometry can lead to difficulties in interpreting the signals, although technicians with specific experience on weldments similar to those to be examined may be able to decipher UT readings in some instances. Similarly, UT is usually not suitable for use with fillet welds and smaller PJP groove welds. CJP groove welds with and without backing bars also give readings that are subject to differing interpretations. Ultrasonic examination may be specified to validate the integrity of CJP groove welds that are subject to tension. Ultrasonic examination has largely replaced radiographic examination for the inspection of critical CJP groove welds.

Radiographic Testing (RT)

Radiographic testing is basically an X-ray film process. To be detected by radiography, a crack must be oriented roughly parallel to the impinging radiation beam, and occupy about $1^1/_2$ percent of the metal thickness along that beam. There are problems with radiographs of fillets, tee, and corner joints, however, because the radiation beam must penetrate varying thicknesses.

Precautions for avoiding radiation hazards interfere with shop work, and equipment and film costs make it the most expensive inspection method. Ultrasonic systems have gradually supplemented and even supplanted radiography.

Radiographic examination has very limited applicability in some applications, such as for HSS fabrication, because of the irregular shape of common joints and the resulting variations in thickness of material as projected onto film. RT can be used successfully for butt splices, but can only provide limited information about the condition of fusion at backing bars near the root corners. The general inability to place either the radiation source or the film inside the HSS means that exposures must usually be taken through both the front and back faces of the section with the film attached to the outside of the back face. Several such shots progressing around the member are needed to examine the complete joint.

PROPER SPECIFICATION OF JOINT TYPE

Selection of Weld Type

The most common weld types are fillet and groove welds. Fillet welds are normally more economical than groove welds and generally should be used in applications for which groove welds are not required. Additionally, fillet welds around the inside of holes or slots require less weld metal than plug or slot welds of the same size, even though the diameters of holes and widths of slots for fillet welds must be larger to accommodate the necessary tilt of the electrode.

Partial joint penetration (PJP) groove welds are more economical than complete joint penetration (CJP) groove welds. When groove welds are required, bevel and V groove welds, which can be flame-cut, are usually more economical than J and U groove welds, which must be air-arc gouged or planed. Also, double-bevel, double-V, double-J, and double-U welds are typically more economical than welds of the same type with single-sided preparation because they use less weld metal, particularly as the thickness of the connection element(s) being welded increases. The symmetry also results in less rotational distortion strain. However, in thinner connection elements, the savings in weld-metal volume may not offset the additional cost of double edge preparation, weld-root cleaning and repositioning. As a general rule of thumb, double-sided joint preparation is normally less expensive than single-sided preparation above 1-in. thickness.

Weld Symbols

For guidance on the proper use of weld symbols, refer to Table 8–2. More extensive information on weld symbols may be found in AWS A2.4 *Standard Symbols for Welding, Brazing, and Nondestructive Examination.*

Available Strength

The available strength of a welded joint is determined in accordance with AISC Specification Section J2.4 and Table J2.5. The calculation of the available strength of a longitudinally loaded fillet weld can be simplified from that given in AISC Specification Table J2.5. For a fillet weld less than or equal to 100 times the weld size in length, the available strength, ϕR_n or R_n/Ω, may be calculated as follows:

$$R_n = 0.6F_{EXX} \times \frac{\sqrt{2}}{2} \times \frac{D}{16} \times l$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where

l = length, in.

D = weld size in sixteenths of an inch

for F_{EXX} = 70 ksi:

LRFD	ASD
$\phi R_n = 1.392Dl$	$R_n/\Omega = 0.928Dl$

When the fillet weld is not longitudinally loaded, the provisions in AISC Specification Section J2.4a may be used to take advantage of the increased strength due to load angle. The maximum strength increase will be for a transversely loaded fillet weld, which is 50 percent stronger than the same fillet weld longitudinally loaded.

Effect of Load Angle

When designing fillet welds, the increased strength due to loading angle may be accounted for by multiplying the available strength of the weld by the following expression:

$$(1.0 + 0.50\sin^{1.5}\theta)$$

where

θ = Loading angle

For transversely loaded welds, $\theta = 90°$. This accounts for a 50 percent increase in weld strength over a longitudinally loaded weld. However, this increased weld strength is accompanied by a decrease in ductility. For a single line weld, the decreased ductility is inconsequential for most applications. However, for weld groups composed of welds loaded at various angles, this change in ductility means that the designer must consider load-deformation compatibility.

CONCENTRICALLY LOADED WELD GROUPS

The load-deformation curves shown in Figure 8-5 highlight the need for consideration of deformation compatibility, since the transversely loaded weld will fracture before the longitudinally loaded weld obtains its full strength.

A simplified procedure for determining the available strength of concentrically loaded fillet weld groups is presented in Table 8–1. In lieu of using this procedure, it is permitted to sum the capacities of individual weld elements, neglecting load-deformation compatibility, when no increase in strength due to the loading angle is assumed.

ECCENTRICALLY LOADED WELD GROUPS

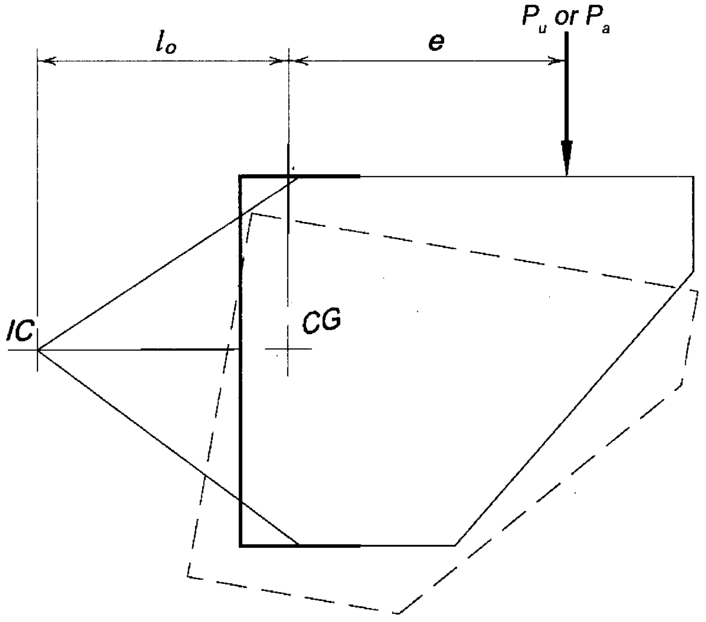
Eccentricity in the Plane of the Faying Surface

Eccentricity in the plane of the faying surface produces additional shear. The welds must be designed to resist the combined effect of the direct shear, P_u or P_a, and the additional shear from the induced moment, $P_u e$ or $P_a e$. Two methods of analysis for this type of eccentricity are the instantaneous center of rotation method and the elastic method.

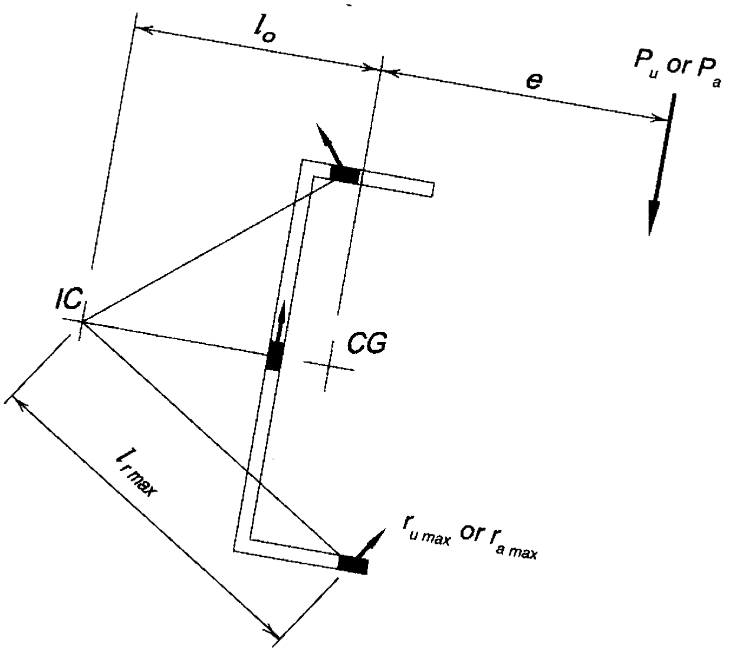
The instantaneous center of rotation method is more accurate, but generally requires the use of tabulated values or an iterative solution. The elastic method is simplified, but may be excessively conservative because it neglects the ductility of the weld group and the potential for load redistribution.

Instantaneous Center of Rotation Method

Eccentricity produces both a rotation and a translation of one connection element with respect to the other. The combined effect of this rotation and translation is equivalent to a rotation about a point defined as the instantaneous center of rotation (IC) as illustrated in Figure 8-4a. The location of the IC depends upon the geometry of the weld group as well as the direction and point of application of the load.



(a) Instantaneous center of rotation (IC)



(b) Forces on weld elements

Figure 8–4. Instantaneous center of rotation method.

The load-deformation relationship for a unit-length segment of the weld, as illustrated in Figure 8-5, is an approximation of the equation presented by Lesik and Kennedy (Lesik and Kennedy, 1990), where

$$P = 0.60F_{EXX}\,(1.0 + 0.50\sin^{1.5}\theta)\,[p\,(1.9 - 0.9p)]^{0.3}$$

where

P = nominal shear strength of the weld segment at a deformation Δ, kips.
F_{EXX} = weld electrode strength, ksi.
θ = load angle measured relative to the weld longitudinal axis, degrees.
p = ratio of element deformation to its deformation at maximum stress.

Unlike the load-deformation relationship for bolts, the strength and deformation of welds are dependent upon the angle θ that the resultant elemental force makes with the axis of the weld element. Load-deformation curves in Figure 8-5 for values of θ = 0°, 15°, 30°, 45°, 60°, 75°, and 90° are shown relative to $P_o = 0.6F_{EXX}$. For further information, see AISC Specification Section J2.4b and its Commentary.

The nominal shear strength of the weld group is governed by Δ_{max} of the weld segment that first reaches its limit, where

$$\Delta_{max} = 1.087w\,(\theta + 6)^{-0.65} \le 0.17w$$

where w is the weld leg size, in.

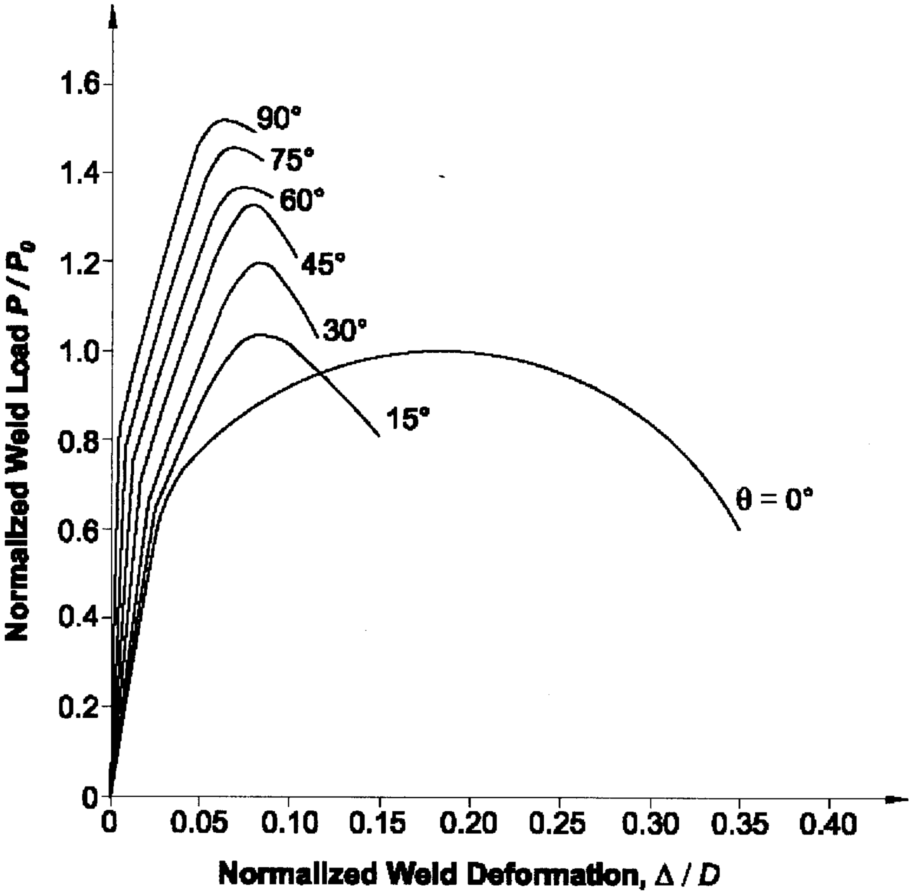


Figure 8–5. Fillet-weld strength as a function of load angle, θ.

The nominal shear strengths of the other unit-length weld segments in the joint can be determined by applying a deformation Δ that varies linearly with distance from the IC. The nominal shear strength of the weld group is, then, the sum of the individual strengths of all weld segments. Because of the non-linear nature of the requisite iterative solution, for sufficient accuracy, a minimum of twenty weld elements for the longest line segment is generally recommended.

The individual resistance of each weld segment is assumed to act on a line perpendicular to a ray passing through the IC and the centroid of that weld segment, as illustrated in Figure 8-4b. If the correct location of the instantaneous center has been selected, the three equations of in-plane static equilibrium ($\Sigma F_x = 0$, $\Sigma F_y = 0$, $\Sigma M = 0$) will be satisfied.

For further information, see Crawford and Kulak (1968) and Butler, Pal, and Kulak (1972).

Elastic Method

For a force applied as illustrated in Figure 8-4, the eccentric force, P_u or P_a, is resolved into a force, P_u or P_a, acting through the center of gravity (CG) of the weld group and a moment, $P_u e$ or $P_a e$, where e is the eccentricity. Each weld element is then assumed to resist an equal share of the direct shear, P_u or P_a, and a share of the eccentric moment, $P_u e$ or $P_a e$, proportional to its distance from the CG. The resultant vectorial sum of these forces, r_u or r_a, is the required strength for the weld.

The shear per linear inch of weld due to the concentric force, r_p, is determined as

LRFD	ASD
$r_p = \dfrac{P_u}{l}$	$r_p = \dfrac{P_a}{l}$

where

l is the total length of the weld in the weld group. To determine the resultant shear per linear inch of weld, r_p must be resolved into horizontal component r_{px} and vertical component r_{py}, where

$$r_{px} = r_p \sin\theta \text{ and } r_{py} = r_p \cos\theta$$

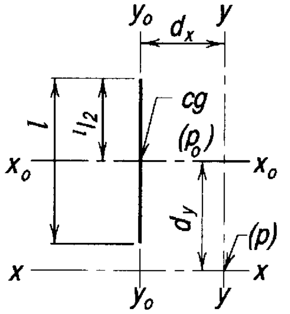
The shear per linear inch of weld due to the moment, $P_u e$ or $P_a e$, is r_m, where

LRFD	ASD
$r_m = \dfrac{P_u ec}{I_p}$	$r_m = \dfrac{P_a ec}{I_p}$

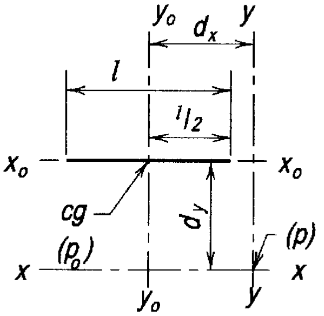
where

c = radial distance from CG to point in weld group most remote from CG, in.

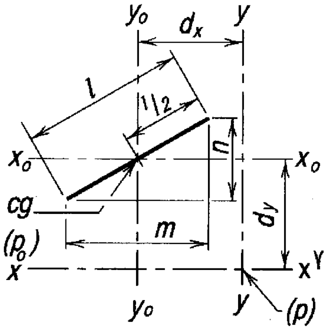
I_p = polar moment of inertia of the weld group, in.^{4} per in.^{2} ($I_p = I_x + I_y$). Refer to Figure 8-6. For section moduli and torsional constants of various welds treated as line elements, refer to Table 5 in Section 7 of Blodgett (1966).



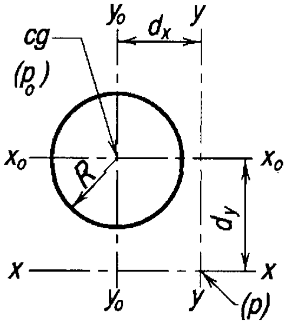
$$I_{xo} = \frac{l^3}{12}$$
$$I_x = \frac{l^3}{12} + l(d_y)^2$$
$$I_{yo} = 0$$
$$I_y = l(d_x)^2$$



$$I_{xo} = 0$$
$$I_x = l(d_y)^2$$
$$I_{yo} = \frac{l^3}{12}$$
$$I_y = \frac{l^3}{12} + l(d_x)^2$$

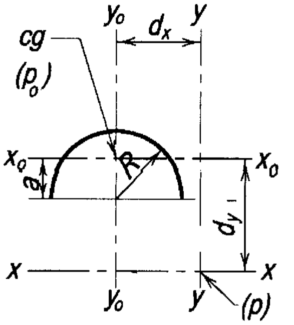


$$I_{xo} = \frac{ln^2}{12}$$
$$I_x = \frac{ln^2}{12} + l(d_y)^2$$
$$I_{yo} = \frac{lm^2}{12}$$
$$I_y = \frac{lm^2}{12} + l(d_x)^2$$



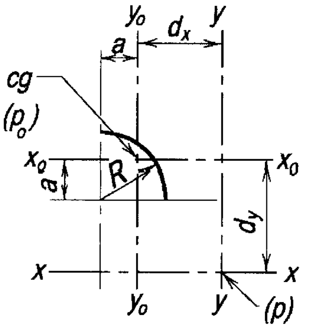
$l = 6.283R$

$$I_{xo} = \pi R^3$$
$$I_x = \pi R^3 + l(d_y)^2$$
$$I_{yo} = \pi R^3$$
$$I_y = \pi R^3 + l(d_x)^2$$



$a = 0.637R$
$l = 3.14R$

$$I_{yo} = \frac{\pi}{2}R^3$$
$$I_y = \frac{\pi}{2}R^3 + l(d_x)^2$$
$$I_{xo} = \left(\frac{\pi}{2} - \frac{4}{\pi}\right)R^3$$
$$I_x = \left(\frac{\pi}{2} - \frac{4}{\pi}\right)R^3 + l(d_y)^2$$



$a = 0.637R$
$l = 3.14R$

$$I_{xo} = \left(\frac{\pi}{4} - \frac{2}{\pi}\right)R^3$$
$$I_x = \left(\frac{\pi}{4} - \frac{2}{\pi}\right)R^3 + l(d_y)^2$$
$$I_{yo} = \left(\frac{\pi}{4} - \frac{2}{\pi}\right)R^3$$
$$I_y = \left(\frac{\pi}{4} - \frac{2}{\pi}\right)R^3 + l(d_x)^2$$

Figure 8–6. Moments of inertia of various weld segments.

To determine the resultant force on the most highly stressed weld element, r_m must be resolved into horizontal component r_{mx} and vertical component r_{my}, where

LRFD	ASD
$r_{mx} = \dfrac{P_u e c_y}{I_p}$ and $r_{my} = \dfrac{P_u e c_x}{I_p}$	$r_{mx} = \dfrac{P_a e c_y}{I_p}$ and $r_{my} = \dfrac{P_a e c_x}{I_p}$

In the above equations, c_x and c_y are the horizontal and vertical components of the radial distance c. Thus, the resultant force, r_u or r_a, is determined as

LRFD	ASD
$r_u = \sqrt{\left(r_{px} + r_{mx}\right)^2 + \left(r_{py} + r_{my}\right)^2}$	$r_a = \sqrt{\left(r_{px} + r_{mx}\right)^2 + \left(r_{py} + r_{my}\right)^2}$

which should be compared against the available strength, found in AISC Specification Table J2.5. For further information, see Higgins (1971).

Eccentricity Normal to the Plane of the Faying Surface

Eccentricity normal to the plane of the faying surface produces tension above and compression below the neutral axis, as illustrated in Figure 8-7 for a bracket connection. The eccentric force, P_u or P_a, is resolved into a direct shear, P_u or P_a, acting at the faying surface of the joint and a moment normal to the plane of the faying surface, $P_u e$ or $P_a e$, where e is the eccentricity. Each unit-length segment of weld is then assumed to resist an equal share of the concentric force, P_u or P_a, and the moment is resisted by tension in the welds above the neutral axis and compression below the neutral axis.

In contrast to bolts, where the interaction of shear and tension must be considered, for welds, shear and tension can be combined vectorially into a resultant shear. Thus, the solu-

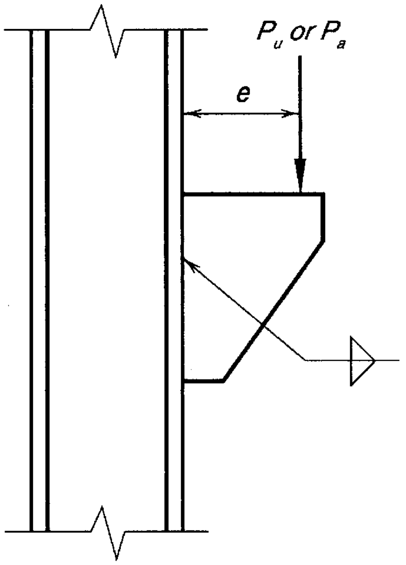


Figure 8–7. Welds subject to eccentricity normal to the plane of the faying surface.

tion of a weld loaded eccentrically normal to the plane of the faying surface is similar to that discussed previously for welds loaded eccentrically in the plane of the faying surface.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

The following other specification requirements and design considerations apply to the design of tension members.

Special Requirements for Heavy Shapes and Plates

For complete-joint-penetration groove welded joints in heavy shapes with a flange thickness exceeding 2 in. or built-up sections consisting of plates with a thickness exceeding 2 in., see AISC Specification Sections A3.1c and Section A3.1d.

Placement of Weld Groups

For the required placement of weld groups at the ends of axially loaded members, see AISC Specification Section J1.7.

Welds in Combination with Bolts or Rivets

For welds used in combination with bolts or rivets, see AISC Specification Section J1.8.

Fatigue

For applications involving fatigue, see AISC Specification Appendix 3.

One-Sided Fillet Welds

When lateral deformation is not otherwise prevented, a severe notch can result at locations of one-sided welds. For the fillet-welded joint illustrated in Figure 8-8, the unwelded side has no strength in tension and a notch may form from the unwelded side. Using one fillet weld on each side will eliminate this condition. This is also true with partial-joint penetration groove welds.

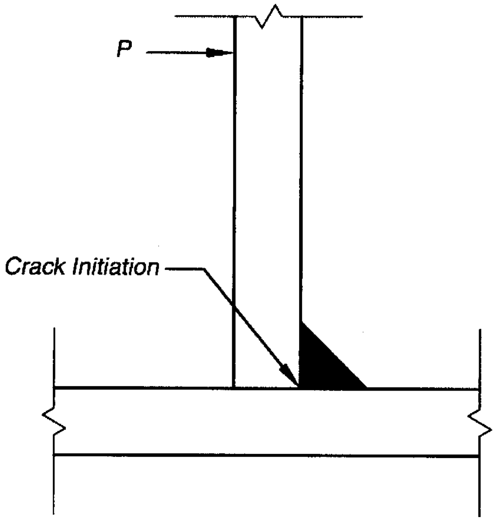


Figure 8–8. Notch effect at one-sided weld.

Welding Considerations and Appurtenances

Clearance Requirements

Clearances are required to allow the welder to make proper welds. Ample room must be provided so that the welder or welding operator may manipulate the electrode and observe the weld as it is being deposited.

In the SMAW process, the preferred position of the electrode when welding in the horizontal position is in a plane forming 30° with the vertical side of the fillet weld being made. However, this angle, shown as angle x in Figure 8-9, may be varied somewhat to avoid contact with some projecting part of the work. A simple rule to provide adequate clearance for the electrode in horizontal fillet welding is that the clear distance to a projecting element should be at least one-half its height (y); distance ($y/2$) in Figure 8-10b.

A special case of minimum clearance for welding with a straight electrode is illustrated in Figure 8-10. The 20° angle is the minimum that will allow satisfactory welding along the bottom of the angle and therefore governs the setback with respect to the end of the beam. If a $^1/_2$-in. setback and $^3/_8$-in. electrode diameter were used, the clearance between the angle and the beam flange could be no less than $1^1/_4$ in. for an angle with a leg dimension w of 3 in., nor less than $1^5/_8$ in. with a w of 4 in. When it is not possible to provide this clearance, the end of the angle may be cut as noted by the optional cut in Figure 8-11 to allow the necessary angle. However, this secondary cut will increase the cost of fabricating the connection.

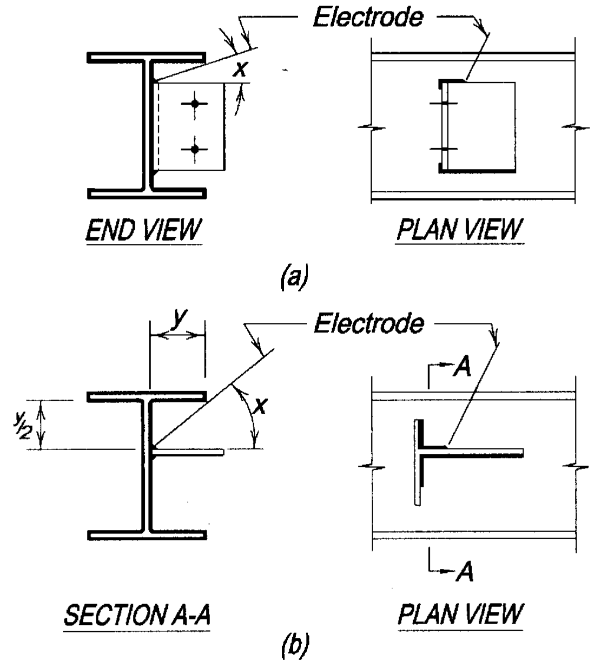


Figure 8–9. Clearances for SMAW welding.

Excessive Welding

The specification of over- or excessive welding will increase the amount of heat input into the parts joined and thereby add to distortion in the joint. Distortion of the joint is caused by three fundamental dimensional changes that occur during and after welding:

1. transverse shrinkage that occurs perpendicular to the weld line,
2. longitudinal shrinkage that occurs parallel to the weld line, and
3. angular change that consists of rotation around the weld line.

If these dimensional changes alter the joint so that it is no longer within fabrication tolerances, the joint may need to be repaired with additional heating to bring the joint back to within fabrication tolerances. This added work will result in expensive repair costs which could have been avoided with appropriately sized welds.

Over-specification of weld size also increases the cost of welding for no structural benefit.

Minimum Shelf Dimensions for Fillet Welds

The recommended minimum shelf dimensions for normal size SMAW fillet welds are summarized in Figure 8-11. SAW fillet welds would require a greater shelf dimension to contain the flux, although auxiliary material can be clamped to the member to provide for this. The dimension *b* illustrated in Figure 8-12 must be sufficient to accommodate the combined dimensional variations of the angle length, cope depth, beam depth, and weld size.

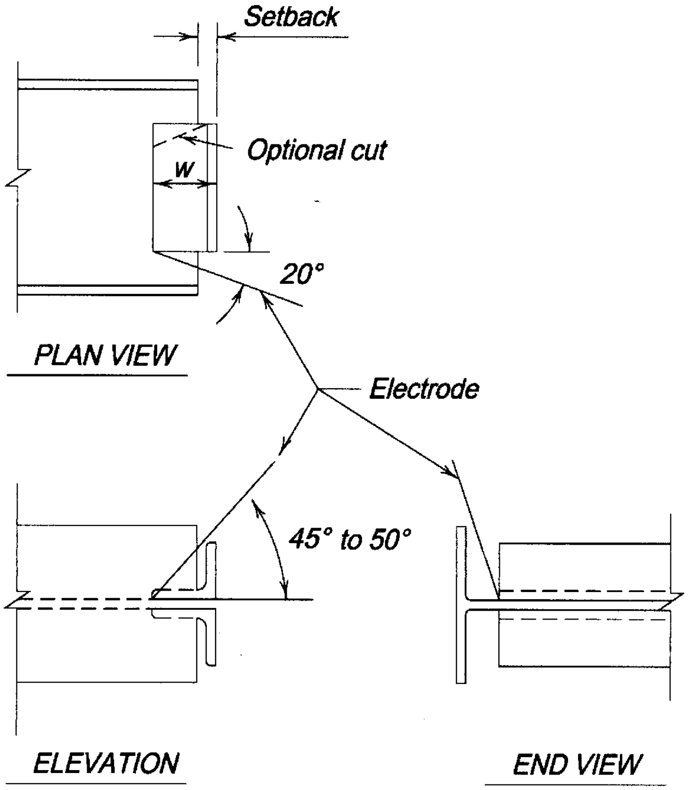


Figure 8–10. Clearances for SMAW welding.

Beam Copes and Weld Access Holes

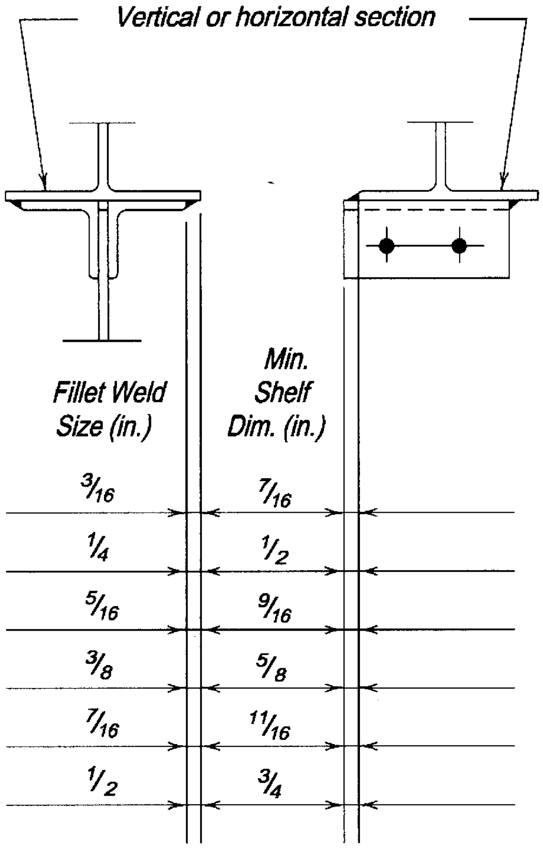
Requirements for beam copes and weld access holes are given in AISC Specification Section J1.6. Weld access holes, as illustrated in Figure 8-14, are used to permit down-hand welding to the beam bottom flange, as well as the placement of a continuous backing bar under the beam top flange. Weld access holes also help to mitigate the effects of weld shrinkage strains and prevent the intersection or close juncture of welds in orthogonal directions. Weld access holes should not be filled with weld metal because doing so may result in a state of triaxial stress under loading.

Corner Clips

Corners of stiffeners and similar elements that fit into a corner should be clipped generously to avoid the lack of fusion that would likely result in that corner. In general, a $^3/_4$-in. clip will be adequate, although this dimension can be adjusted to suit conditions, such as when the fillet radius is larger or smaller than that for which a $^3/_4$-in. clip is appropriate. For further information, see Butler, Pal, and Kulak (1972) and Blodgett (1980).

Backing Bars

Backing bars, illustrated in Figure 8-13, should be of approved weldable material as specified in AWS D1.1 Section 5.2.2.2. Per AWS D1.1, backing bars on groove-welded joints



Fillet Weld Size (in.)	Min. Shelf Dim. (in.)
$^3/_{16}$	$^7/_{16}$
$^1/_4$	$^1/_2$
$^5/_{16}$	$^9/_{16}$
$^3/_8$	$^5/_8$
$^7/_{16}$	$^{11}/_{16}$
$^1/_2$	$^3/_4$

Figure 8–11. Recommended minimum shelf dimensions for SMAW fillet welds.

must be continuous or fully spliced to avoid stress concentrations or discontinuities and should be thoroughly fused with the weld metal. Backing bar removal is addressed in AISC Specification Section J2.6 and AWS D1.1.

Spacer Bars

Spacer bars, illustrated in Figure 8-13, must be of the same material specification as the base metal, per AWS D1.1 Section 5.2.2.3. This can create a procurement problem, since small tonnage requirements may make them difficult to obtain in the specified ASTM designation.

Weld Tabs

To obtain a fully welded cross section, the termination at either end of the joint must be of sound weld metal. Weld tabs, illustrated in Figure 8-13, should be of approved weldable material as specified in AWS D1.1 Section 5.2.2.1. Various configurations of weld tabs are illustrated in Figure 8-14, including flat-type weld tabs, which are normally used with bevel and V groove welds, and contour-type weld tabs, which are normally used with J and U groove welds. Weld-tab removal is addressed in AWS D1.1. Frequently, the backing bar can be extended to serve as the weld tab.

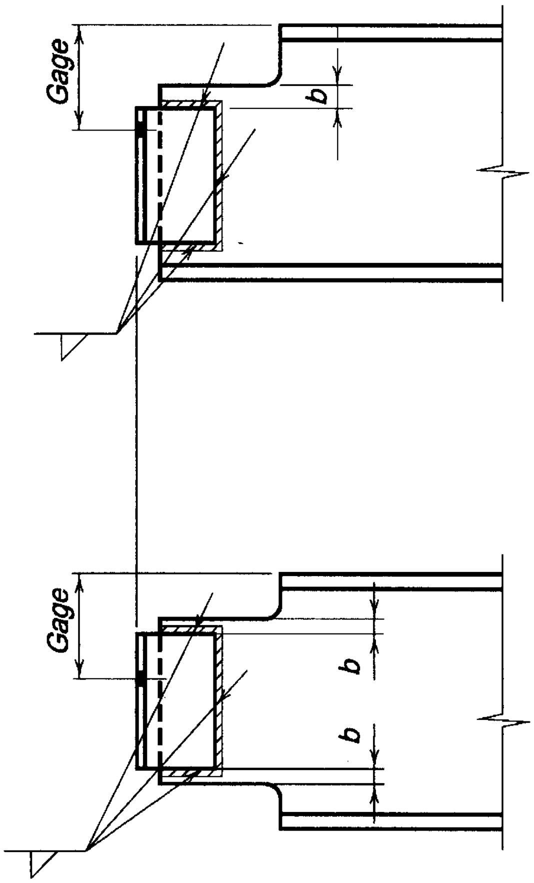
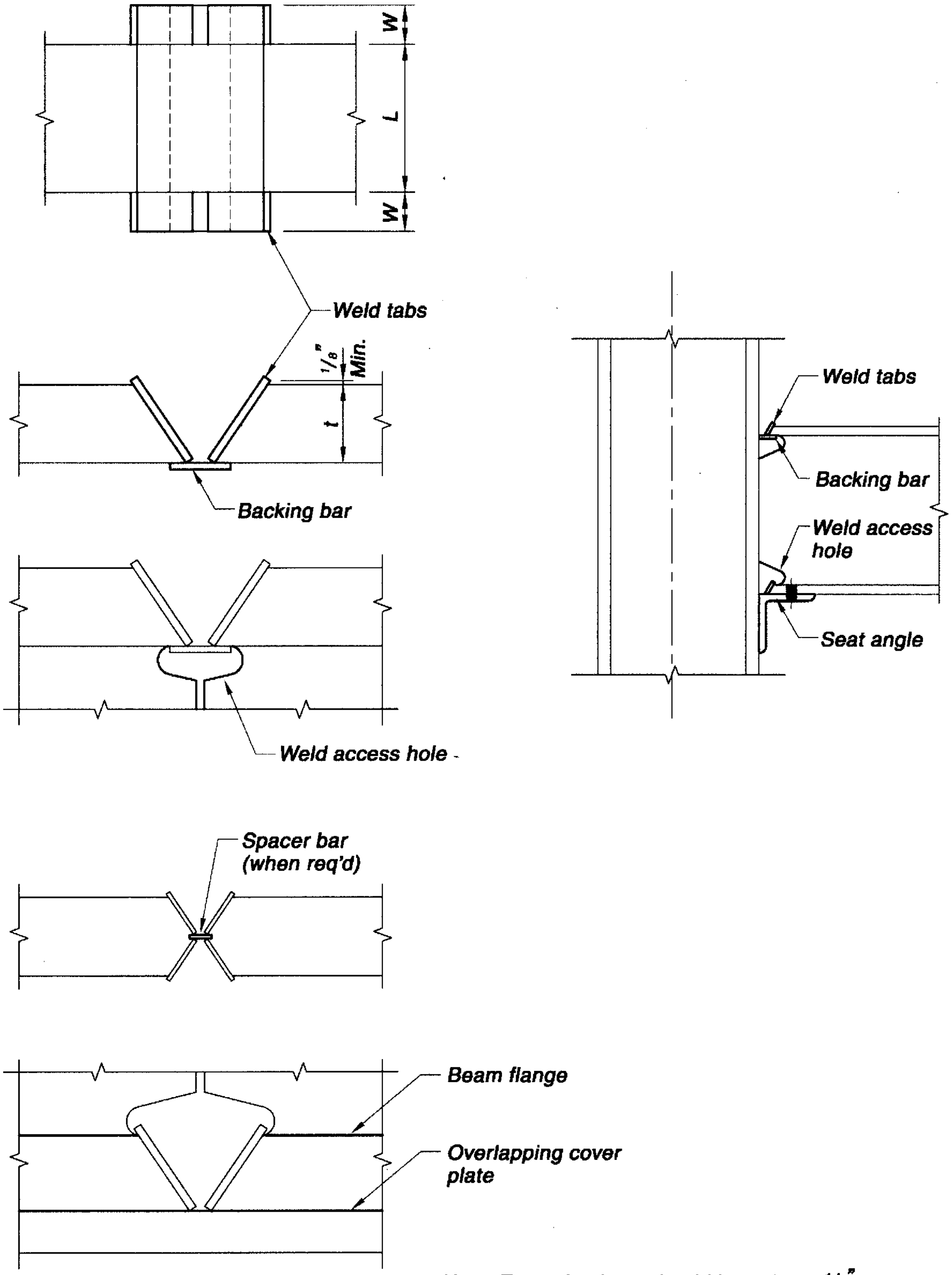


Figure 8–12. Illustration of shelf dimensions for fillet welding.

Tack Welds

Tack welds placed as shown in Figure 8-15a should be avoided as they may cause notches. An improved detail is as shown in Figure 8-15b, with the tack welds placed where they will be consumed in the final welded joint.



Note: Extension bars should be at least $1/4$" thick to reduce hazard of weld "blow through"

Figure 8–13. Illustration of backing bars, spacer bars, weld tabs, and other fittings for welding.

Lamellar Tearing

Figures 8-16 and 8-17 illustrate preferred welded joint selection and connection configurations for avoiding susceptibility to lamellar tearing. Refer to the discussion "Avoiding Lamellar Tearing" in Part 2.

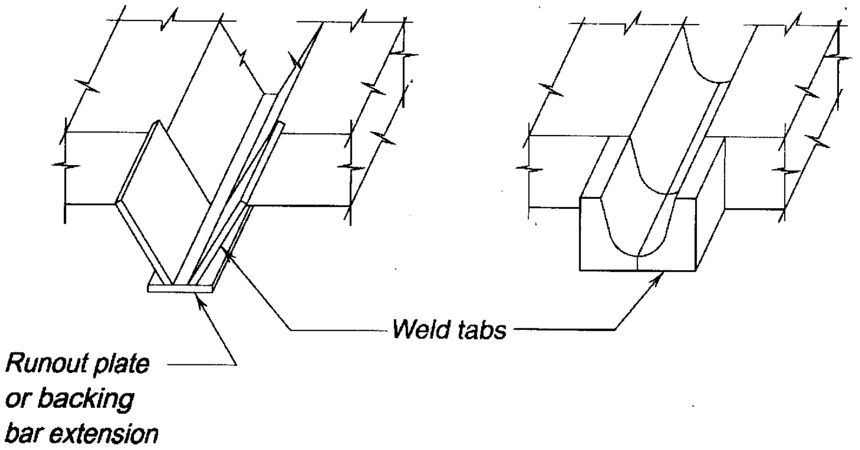
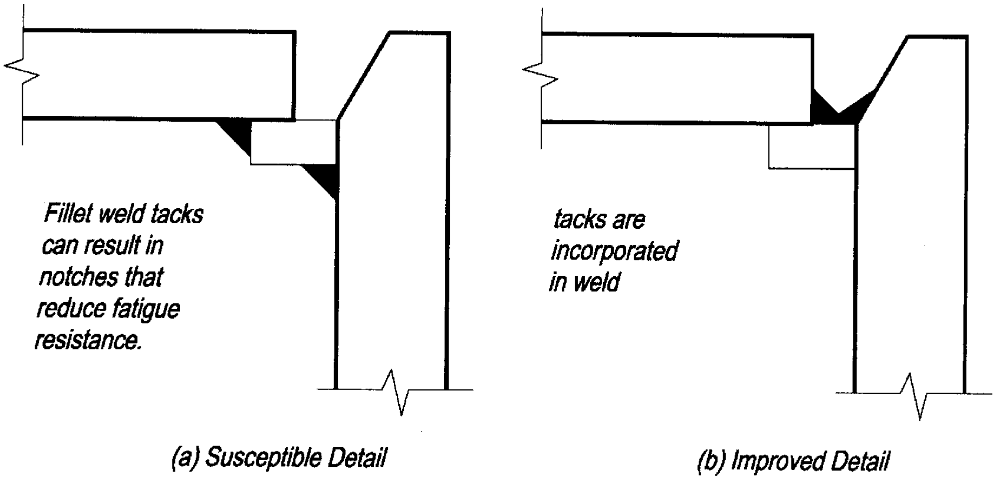


Figure 8–14. Illustration of weld tabs.



(a) Susceptible Detail

(b) Improved Detail

Figure 8–15. Backing bar tack welding.

Prior Qualification of Welding Procedures

Evidence of prior qualification of welding procedures, welders, welding operators, or tackers may be accepted at the discretion of the owner's designated representative for design, resulting in significant cost savings. Fabricators that participate in the AISC Quality Certification Program have the experience and documentation necessary to assure that such prior qualifications could be accepted. For more information about the AISC Quality Certification Program, visit www.aisc.org.

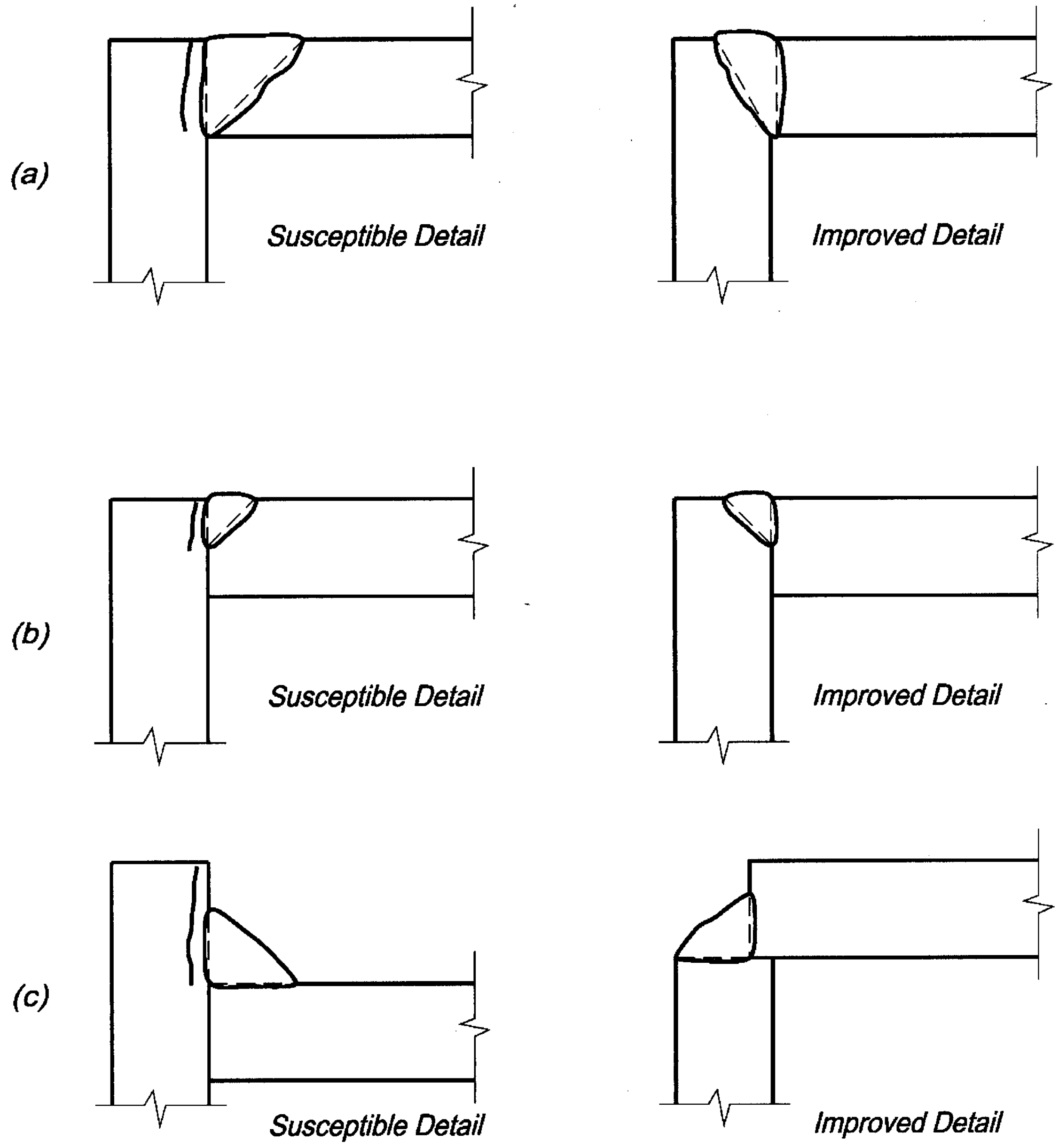


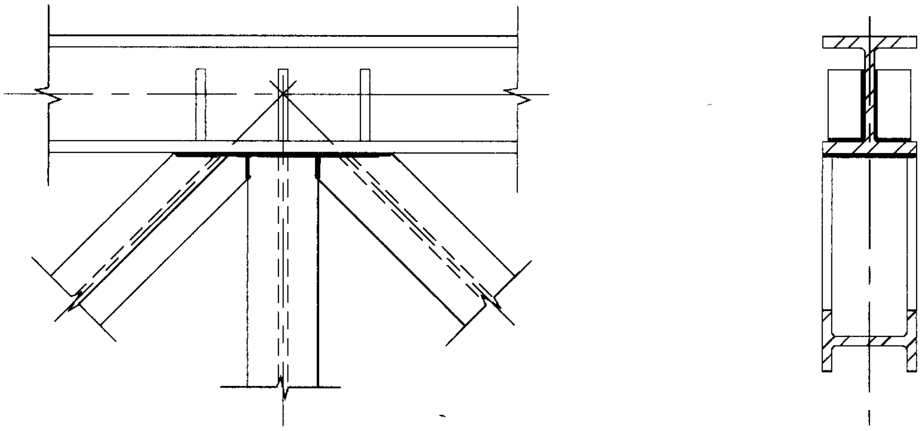
Figure 8–16. Susceptible and improved details to reduce the incidence of lamellar tearing.

Painting Welded Connections

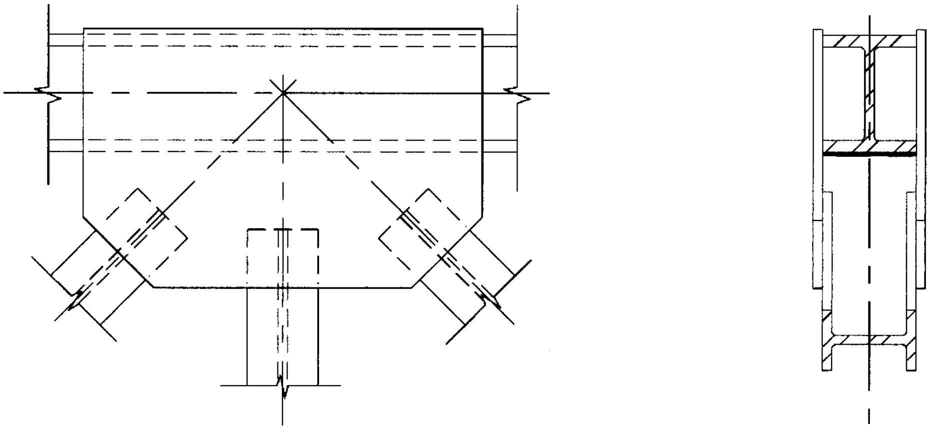
Paint is normally omitted in areas to be field-welded, per AISC Specification Section M3.5. Note that this requirement does not generally apply to shop-assembled connections, because painting is normally done after the welds are made. When required, the small paint-free areas can generally be identified with a general note (e.g., "no paint on OSL of connection angles," where OSL stands for outstanding leg).

WELDING CONSIDERATIONS FOR HSS

Flare welds are more common in HSS because of the increasing likelihood that the HSS corner is a part of the welded joint. A common flare bevel configuration which occurs when equal width sections are joined is illustrated in Figure 8-18. The easiest arrangement for welding occurs with equal wall thickness sections. However, when the corner radius



Susceptible Detail



Improved Detail

Figure 8–17. Susceptible and improved details to avoid intersecting welds with high restraint.

increases due to wall thickness or manufacturing tolerances, the root gap may need to be adjusted by profile shaping, building out with weld metal, or by use of backing. See Figures 8-18 and 8-19.

HSS Welding Requirements in AWS D1.1

AWS uses the terminology "tubular" for all hollow members including pipe, hollow structural sections, and fabricated box sections. The following sections in AWS D1.1 2004 apply to welded HSS to HSS connections:

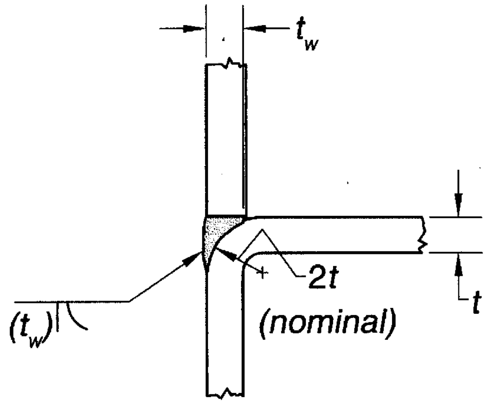


Figure. 8–18. Flare bevel weld, equal width HSS weld joint.

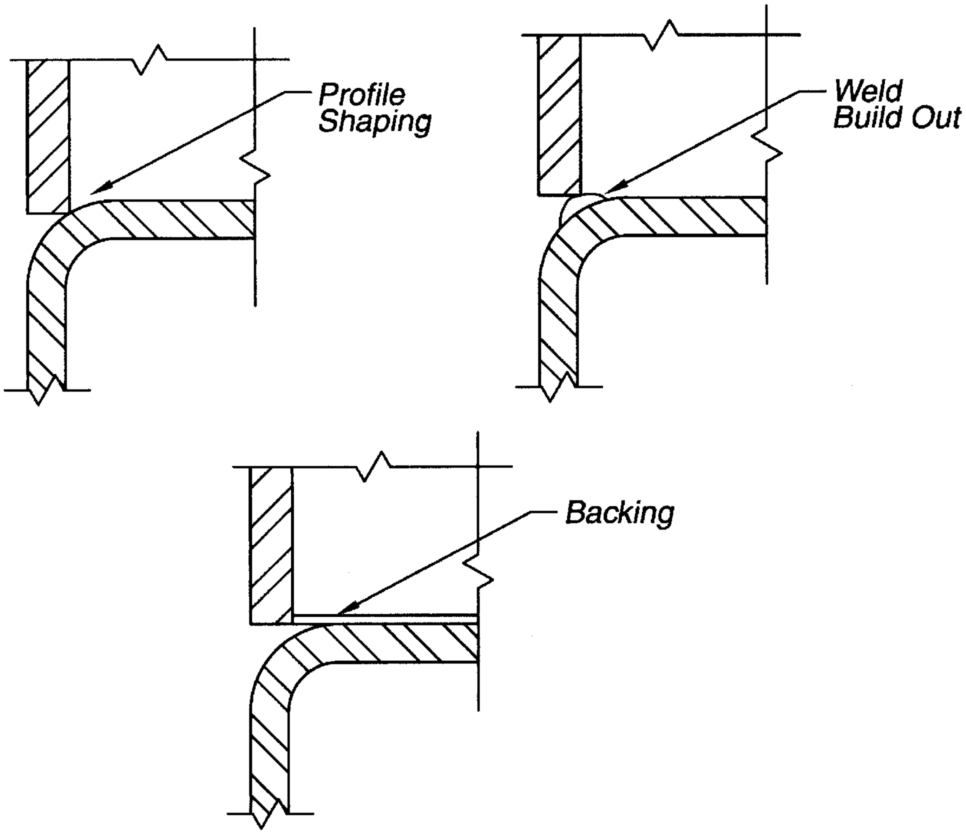


Figure 8–19. Welding methods accounting for the HSS corner radius.

Section 2, Part D

As explained in AWS D1.1 Commentary Section C2.20, "In commonly used types of tubular connections, the weld itself may not be the factor limiting the strength of the joint. Such limitations as local failure (punching shear), general collapse of the main member, and lamellar tearing are discussed because they are not adequately covered in other codes." Because of these various failure modes, the design of HSS-to-HSS connections must be part of the member sizing process. The members selected must be capable of transmitting the required strength or adequate reinforcement must be shown on the design documents.

Differences in the relative stiffness across HSS walls loaded normal to their surface can make the load transfer highly non-uniform. To prevent progressive failure and to ensure ductile behavior of the joint, minimum welds must be provided in T-, Y-, and K-connections to transmit the factored load in the branch or web member. For normal building applications, fillet welds and partial joint penetration welds (PJP) can be used.

While Part D deals primarily with HSS-to-HSS connections, some of these provisions are applicable to welded attachments that deliver a load normal to the wall of a tubular member.

Section 3

AWS Fig. 3.2 shows prequalified fillet weld details for tubular joints that differ from details for non-tubular skewed T-joints. These details will provide minimum weld strength needed to ensure ductile joint behavior.

AWS Fig. 3.3 (10) shows the joint detail and the effective throat for a flare bevel PJP groove weld that is commonly used for welding connection material to the face of an HSS. Groove weld joint details for HSS are designed to accommodate both the geometry of the section and the lack of access to the back side of the joint.

Table 8–2 (AWS Fig. 3.5) shows various PJP groove welded HSS joint details and AWS Figs. 3.6, 3.8, 3.9, and 3.10 show CJP groove welded HSS joint details. The joint preparation and weld sizing are complex and critical to obtain a sound weld. These details also provide the weld strength needed to ensure ductile joint behavior.

Section 4

Qualification covers the requirements for qualification testing of Welding Procedure Specifications (WPS, see p. 8-3) and performance testing of the welder's ability to produce sound welds. HSS connections may not always meet the requirements for a prequalified WPS because of unique geometry, connection access, or for other reasons. This section also gives the requirements for a Procedure Qualification Record (PQR), which is the basis for qualifying a WPS.

The performance testing of welders and welding operators considers process, material thickness, position, non-tubular, or tubular joint access. AWS Tables 4.9 and 4.10 list the required qualifications needed for each type of joint. Most welders are qualified for a particular process and position-in-plate (non-tubular) joints. These qualifications will allow the welder to make similar fillet, PJP groove, and backed CJP welds in tubular members. However, certain types of tubular connections, such as unbacked T-, Y-, and K-connections, require special welder certifications because the lack of access to the back of the joint, the position of the connection, and the access to the connection require special skill to produce a sound connection.

Section 5

Fabrication covers the requirements for the preparation, assembly, and workmanship of welded steel structures. AWS Table 5.5 Tubular Root Opening Tolerances gives the acceptable fitup for unbacked groove welds. AWS Table 5.8 Minimum Fillet Weld Size and Section 2.24.1.3 give the minimum weld pass size based on material thickness and process.

Section 6

Inspection contains all of the requirements for the inspector's qualifications and responsibilities, acceptance criteria for discontinuities, and procedures for non-destructive examination (NDE). AWS D1.1 considers fabrication/erection inspection and testing a separate function from verification inspection and testing. Fabrication/erection inspection and testing is usually the responsibility of the contractor and is performed as appropriate prior to assembly, during assembly, during welding, and after welding to ensure the requirements of the contract documents are met. Verification inspection and testing are the prerogatives of the owner. The extent of NDE and verification inspection must be specified in the contract documents.

If non-destructive testing other than visual is not specified in the contract documents, but is subsequently requested, the owner is responsible, per AWS D1.1, for all costs associated with this testing including handling, surface preparation, and repairs (if required).

The inspection covers WPS qualification, equipment, welder qualification, joint preparation, joint fitup, welding techniques, and weld size length and location. It is especially important when inspecting HSS-to-HSS joints that joint preparation and fitup be checked prior to welding.

In addition to inspecting the above items, AWS requires all welds to be visually inspected for conformance to the standards in AWS Table 6.1 Visual Acceptance Criteria.

Four types of non-destructive testing can be used to supplement visual inspection. They are penetrant testing (PT), magnetic particle testing (MT), radiographic testing (RT), and ultrasonic testing (UT).

The AWS UT acceptance criteria for non-HSS type groove welds starts at $^5/_{16}$-in. thick material. The procedures for HSS T-, Y-, and K- connections have a minimum applicable thickness of $^1/_2$ in, and diameter of $12^3/_4$ in. AWS does, however, make provision for qualifying UT procedures for smaller size applications. It is possible to UT portions of butt-type splices with backing bars using the non-HSS criteria, however, the corners of rectangular HSS cannot be inspected.

AWS D1.1 does make provision for using alternate acceptance criteria based upon an evaluation of suitability for service using past experience, experimental evidence, or engineering analysis. This can be especially important when deciding if and how to make any repairs.

Weld Sizing for Uneven Distribution of Loads

The connection strength for a member welded normal to a HSS wall is a function of the geometric parameters of the connected members and is often less than the full strength of the member. When limited by geometry, the available strength cannot be increased by increasing the weld strength. Due to the varying relative flexibility of the HSS wall loaded normal to its surface and the axial stiffness of the connected member, the transfer of load along the weld line is highly non-uniform. To prevent progressive failure, or "unzipping" of the weld, it is important to provide adequate welds to maintain ductile behavior of the joint.

Welds that satisfy this ductility requirement can be proportioned for the required strength using an effective width criteria similar to that used for checking the axial strength of the branch member or plate. For effective weld length of HSS-to-HSS connections, refer to AISC Specification Sections K2-2 and K2-3.

An alternative to the effective length procedure is the use of the prequalified fillet and PJP groove weld details in AWS D1.1 that are sized to ensure ductile behavior. In addition, fillet welds with an effective throat of 1.1 times the thickness of the branch member can be used. Either of these two alternatives will, in most cases, be conservative.

Detailing Considerations

1. Butt joints will require a groove weld detail. Where possible the joint should be a pre qualified PJP groove weld sized for actual load or a CJP groove weld with steel backing.
2. T-, Y-, and K-connections should, where possible, use either fillet welds or PJP groove welds sized for the design forces and checked for the minimum size needed to ensure duc tile joint behavior. Where CJP welds are required, joint details using steel backing should be used whenever possible. For a detailed discussion of various types of backing and the advantages of using backing, see J.W. Post (1990).

DESIGN TABLES

Table 8–1. Coefficients, *C*, for Concentrically Loaded Weld Group Elements

Concentrically loaded fillet weld groups must consider the effect of loading angle and deformation compatibility on weld strength.

By multiplying the appropriate values of *C* from Table 8–1 by the available strength of each weld element, an effective strength is determined for each weld element. The available strength of the weld group can be determined by summing the effective strengths of all of the elements in a weld group. It should be noted that this table is to be entered at the largest load angle on any weld in the weld group. For the weld group shown in Figure 8-20, this is calculated as:

LRFD	ASD
$\phi R_w = (D)(1.392)\left[1.5(1)+1.29(1.41)+0.825(1)\right]$ $= 5.77D$	$R_w/\Omega = (D)(0.928)\left[1.5(1)+1.29(1.41)+0.825(1)\right]$ $= 3.85D$

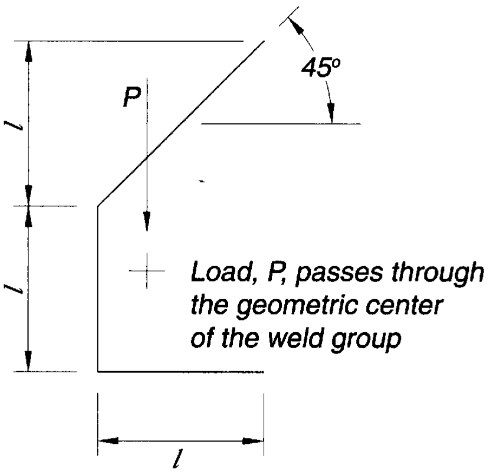


Figure 8–20. Concentrically loaded weld group.

Table 8–2. Prequalified Welded Joints

The prequalified welded joints details given in AWS D1.1 and Table 8–2 provide joint geometries, such as root openings, angles, and clearances (see Figures 8-21 and 8-22) that will permit the deposition of sound weld material. Prequalified welded joints are not, in themselves, adequate consideration of welded design details and the other provisions in AWS D1.1 must be satisfied as they are referenced in AISC Specification Section J2.2. The design and detailing for successful welded construction requires consideration of factors which include, but are not limited to, the magnitude, type, and distribution of forces to be transmitted, access, restraint against weld shrinkage, thickness of connected materials, residual stress, and distortion.

The designations such as B-L1a, B-U2, and B-P3 are those used in AWS D1.1. Note that lowercase letters (e.g., a, b, c, etc.) are often used to differentiate between joints that would otherwise have the same joint designation. These prequalified welded joints are limited to those made by the SMAW, SAW, GMAW (except short circuit transfer), and FCAW procedures. Small deviations from dimensions, angles of grooves, and variation in depth of groove joints are permissible within the tolerances given.

In general, all fillet welds are prequalified, provided they conform to the requirements in AWS D1.1. Groove welds are classified using the conventions indicated in the tables. Welded joints other than those prequalified by AWS may be qualified, provided they are tested and qualified in accordance with AWS D1.1.

Table 8–3. Electrode Strength Coefficient C_1

Electrode strength coefficients, C_1, which can be used to adjust the tabulated values of Tables 8-4 through 8-11 for electrodes other than E70XX, are given in Table 8–3. Note that this coefficient includes an additional reduction factor of 0.90 for E80 and E90 electrodes and 0.85 for E100 and E110; this accounts for the uncertainty of extrapolation to these higher-strength electrodes.

Tables 8–4 through 8–11. Coefficients *C* for Eccentrically Loaded Weld Groups

Tables 8-4 through 8-11 employ the instantaneous center of rotation method in accordance with AISC Specification Section J2.4 for the weld patterns and eccentric conditions indicated and inclined loads at 0°, 15°, 30°, 45°, 60°, and 75°. The tabulated non-dimensional coefficient, *C*, represents the effective strength of the weld group in resisting the eccentric shear force.

When Analyzing a Known Weld Group Geometry

For any of the weld group geometries shown, the available strength, ϕR_n or R_n/Ω, of the eccentrically loaded weld group is determined by

$$R_n = CC_1Dl$$
$$\phi = 0.75 \qquad \Omega = 2.00$$

where

C = tabular value

C_1 = electrode coefficient from Table 8–3

D = number of sixteenths-of-an-inch in the weld size

l = length of the reference weld, in.

In developing these tables, the instantaneous center of rotation method was used, with a convergence criterion of less than $^1/_2$ percent and considering deformation compatibility of adjacent weld elements. The first row in each table ($a = 0$) gives the available strength of a concentrically loaded weld group in accordance with AISC Specification Section J2.4. Linear interpolation within a given table between adjacent a and k values is permitted.

Straight-line interpolation between values for loads at different angles may be significantly unconservative. Either a direct analysis should be performed or the values for the next lower angle increment in the tables should be used for design. For weld group patterns not treated in these tables, a direct analysis is required if the instantaneous center of rotation method is to be used.

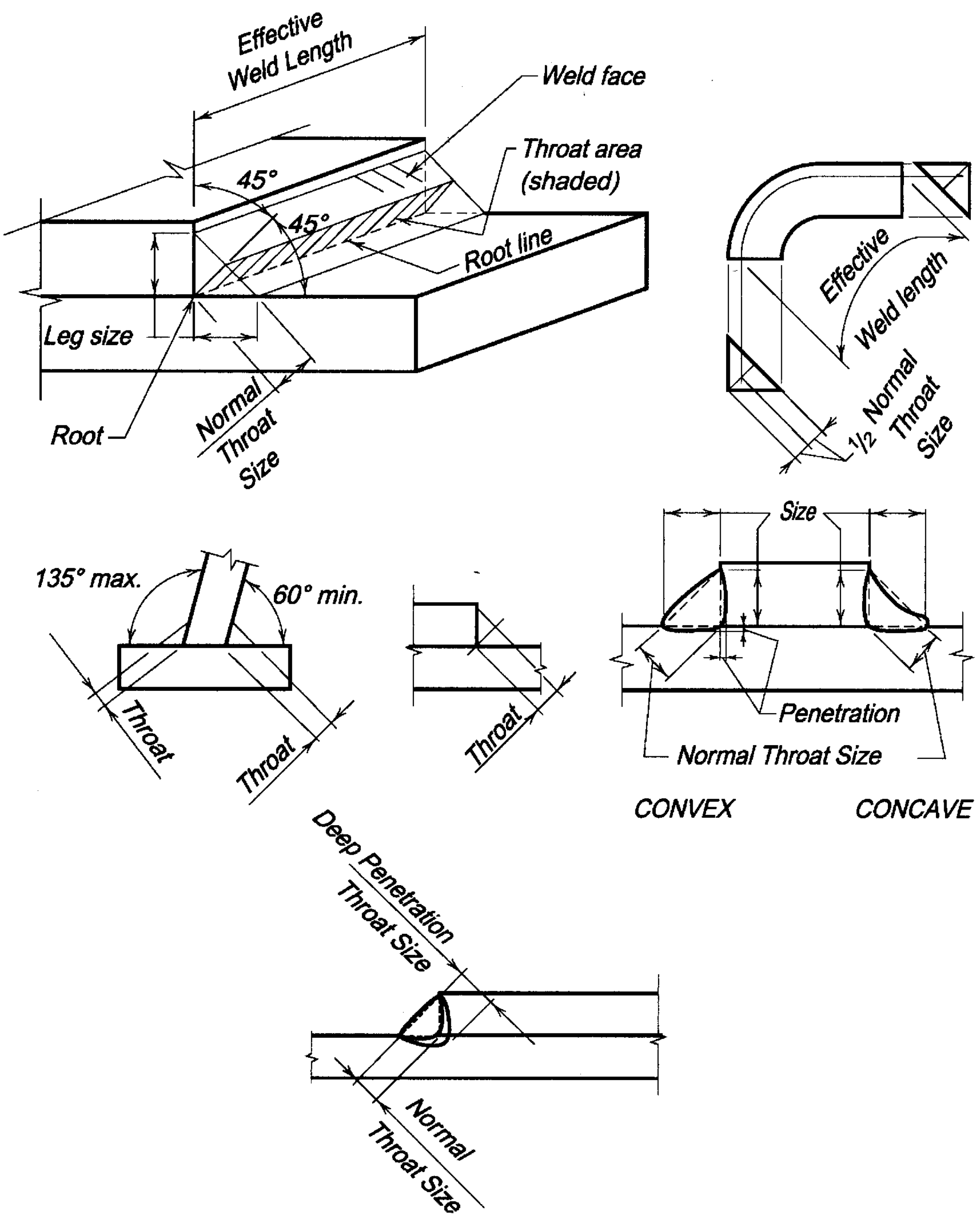
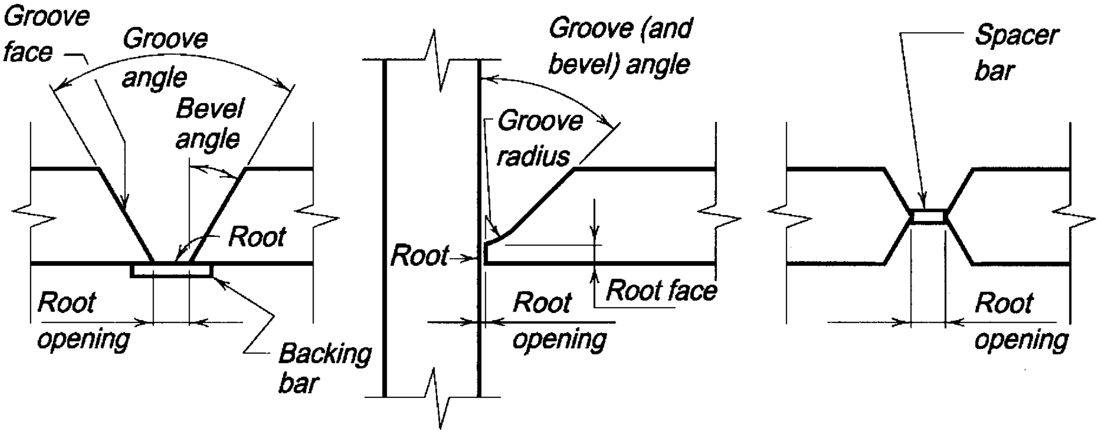
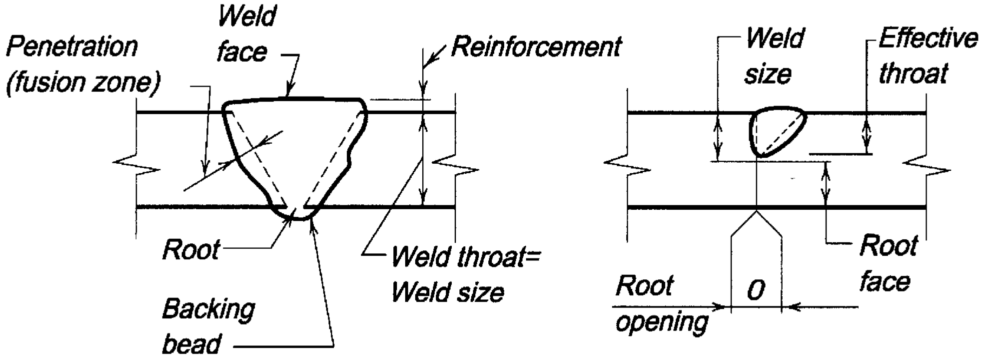


Figure 8–21. Fillet weld nomenclature.

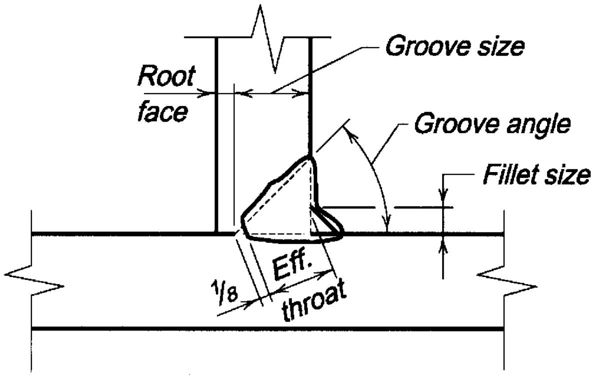


PREPARATION



COMPLETE-JOINT-PENETRATION

PARTIAL-JOINT-PENETRATION



PARTIAL-JOINT-PENETRATION

(When Reinforcing Fillet is Specified)

Figure 8–22. Groove weld nomenclature.

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Table 8–1
Coefficients, *C*, for Concentrically Loaded Weld Group Elements

Load angle on weld element, degrees	Largest load angle on and weld group element, degrees						
	90	75	60	45	30	15	0
0	0.825	0.849	0.876	0.909	0.948	0.994	1.00
15	1.02	1.04	1.05	1.07	1.06	0.883	
30	1.16	1.17	1.18	1.17	1.10		
45	1.29	1.30	1.29	1.26			
60	1.40	1.40	1.39				
75	1.48	1.47					
90	1.50						

Table 8–2
Prequalified Welded Joints

Symbols for Joint Types

B	butt joint	BC	butt or corner joint
C	corner joint	TC	T- or corner joint
T	T-joint	BTC	butt, T-, or corner joint

Symbols for Base Metal Thickness and Penetration

L	limited thickness, complete-joint-penetration
U	unlimited thickness, complete-joint-penetration
P	partial-joint-penetration

Symbols for Weld Types

1	square-groove	6	single-U-groove
2	single-V-groove	7	double-U-groove
3	double-V-groove	8	single-J-groove
4	single-bevel-groove	9	double-J-groove
5	double-bevel-groove	10	flare-bevel-groove

Symbols for Welding Processes if not Shielded Metal Arc Welding (SMAW):

S	submerged arc welding (SAW)
G	gas metal arc welding (GMAW)
F	flux cored arc welding (FCAW)

Symbols for Welding Positions

F	flat
H	horizontal
V	vertical
OH	overhead

Symbols for Joint Designation

The lower case letters (e.g., a, b, c, d, etc.) are used to differentiate between joints that would otherwise have the same joint designation.

Symbols for Dimensions

R	Root opening
α, β	Groove angles
f	Root face
r	J- or U-groove radius
S, S_1, S_2	PJP groove weld depth of groove
E, E_1, E_2	PJP groove weld sizes corresponding to S, S_1, S_2, respectively

Notes to Prequalified Welded Joints

1 Not prequalified for gas metal arc welding (GMAW) using short circuiting transfer nor GTAW. Refer to AWS D1.1 Annex A.

2 Joint is welded from one side only.

3 Cyclic load application limits these joints to the horizontal welding position. Refer to AWS D1.1 Section 2.17.2.

4 Backgouge root to sound metal before welding second side.

5 SMAW joints may be used for prequalified GMAW (except GMAW-S) and FCAW.

6 Minimum effective throat thickness (E) as shown in Specification Table J2.3; S as specified on drawings.

7 If fillet welds are used in buildings to reinforce groove welds in corner and T-joints, they shall be equal to $\frac{1}{4}T_1$, but need not exceed $\frac{3}{8}$ in. Groove welds in corner and T-joints of cyclically loaded structures shall be reinforced with fillet welds equal to $\frac{1}{4}T_1$, but need not exceed $\frac{3}{8}$ in.

8 Double-groove welds may have grooves of unequal depth, but the depth of the shallower groove shall be no less than one-fourth of the thickness of the thinner part joined.

9 Double-groove welds may have grooves of unequal depth, provided these conform to the limitations of Note 6. Also, the effective throat thickness (E) applies individually to each groove.

10 The orientation of the two members in the joints may vary from 135° to 180° for butt joints, or 45° to 135° for corner joints, or 45° to 90° for T-joints.

11 For corner joints, the ouside groove preparation may be in either or both members, provided the basic groove configuration is not changed and adequate edge distance is maintained to support the welding operations without excessive edge melting.

12 Effective throat thickness (E) is based on joints welded flush.

Table 8–2 (continued)
Prequalified Welded Joints

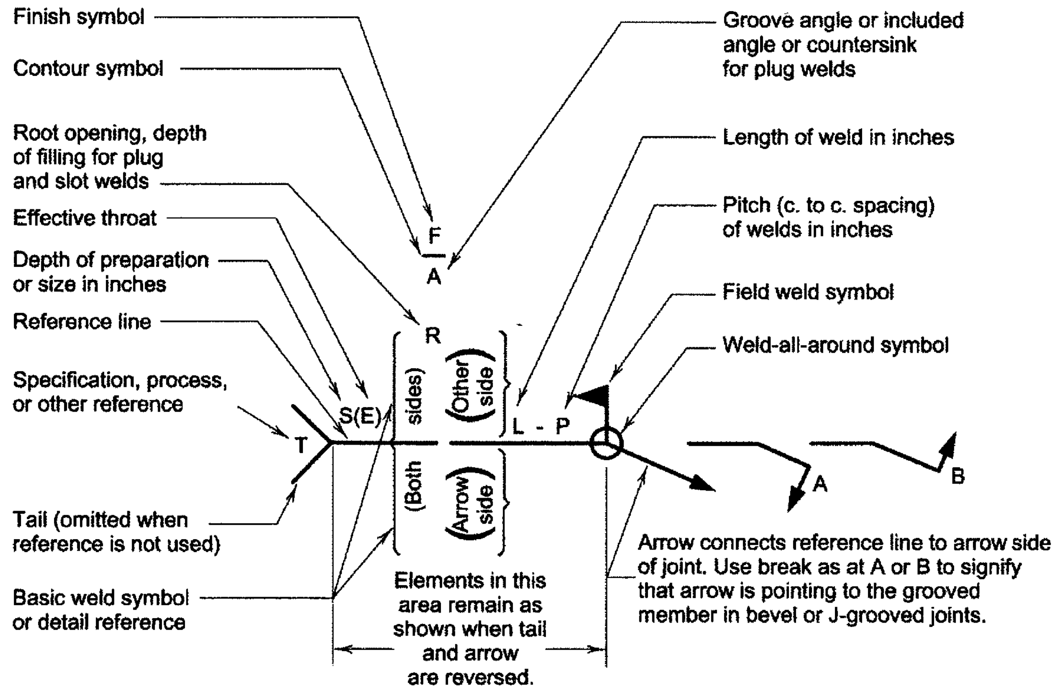
Basic Weld Symbols

Back	Fillet	Plug or Slot	Groove or Butt						
			Square	V	Bevel	U	J	Flare V	Flare Bevel
									

Supplementary Weld Symbols

Backing	Spacer	Weld All Around	Field Weld	Contour		For other basic and supplementary weld symbols, see AWS A2.4
				Flush	Convex	
						

Standard Location of Elements of a Welding Symbol



Note:

Size, weld symbol, length of weld, and spacing must read in that order, from left to right, along the reference line. Neither orientation of reference nor location of the arrow alters this rule.

The perpendicular leg of , , , , weld symbols must be at left.

Arrow and other side welds are of the same size unless otherwise shown. Dimensions of fillet welds must be shown on both the arrow side and the other side symbol.

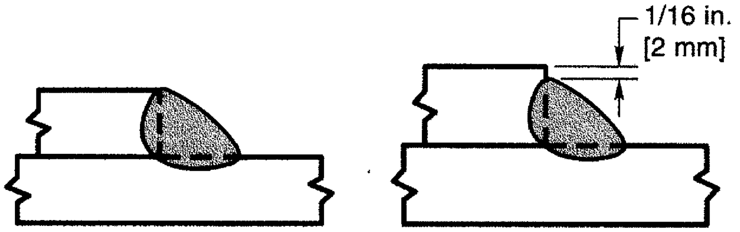
The point of the field weld symbol must point toward the tail.

Symbols apply between abrupt changes in direction of welding unless governed by the "all around" symbol or otherwise dimensioned.

These symbols do not explicitly provide for the case that frequently occurs in structural work, where duplicate material (such as stiffeners) occurs on the far side of a web or gusset plate. The fabricating industry has adopted this convention: that when the billing of the detail material discloses the existence of a member on the far side as well as on the near side, the welding shown for the near side shall be duplicated on the far side.

FILLET

Table 8–2 (continued)
Prequalified Welded Joints Fillet Welds



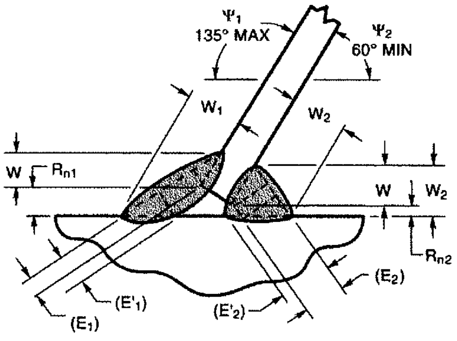
BASE METAL LESS THAN 1/4 in. [6 mm] THICK

(A)

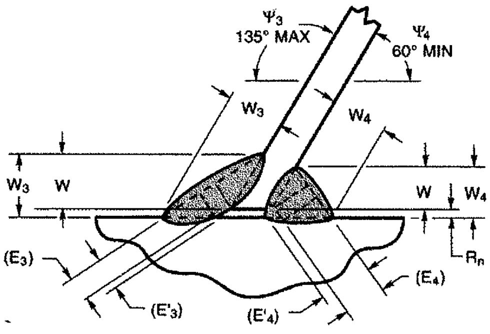
BASE METAL 1/4 in. [6 mm] OR MORE IN THICKNESS

(B)

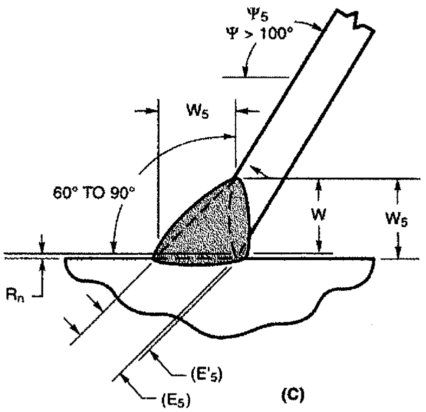
MAXIMUM DETAILED SIZE OF FILLET WELD ALONG EDGES



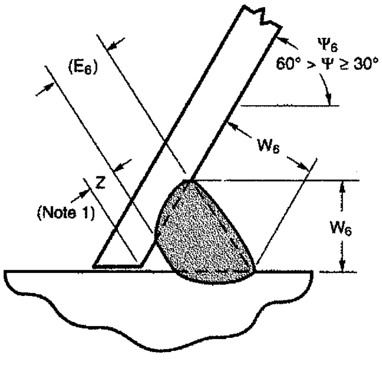
(A)



(B)



(C)



(D)
(See Note 2)

Notes:

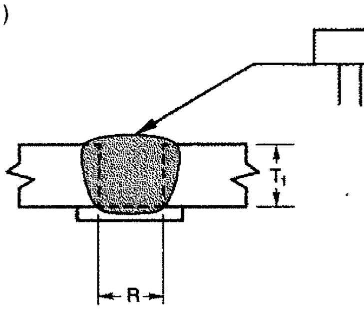
1. (E_n), (E'_n) = Effective throat thickness dependant on magnitude of root opening (R_n). Refer to AWS D1.1 Section 5.22.1. Subscript *n* represents 1, 2, 3, 4, or 5.
2. t = thickness of thinner part.
3. Not prequalified for gas metal arc welding (GMAW) using short circuit transfer nor GTAW. Refer to AWS D1.1 Annex A for GMAW-S.
4. Figure D. Apply Z loss dimension of AWS D1.1 Table 2.2 to determine effective throat thickness.
5. Figure D. Not prequlaified for angles under 30°. For welder qualifications see AWS D1.1 Table 4.8.
6. Angles under 60° are permissible, however, if the weld is considered to be a partial-joint-penetration groove weld.

CJP

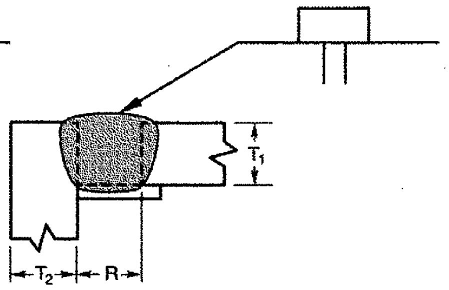
Table 8–2 (continued)

Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Square-groove weld (1)
Butt joint (B)
Corner joint (C)



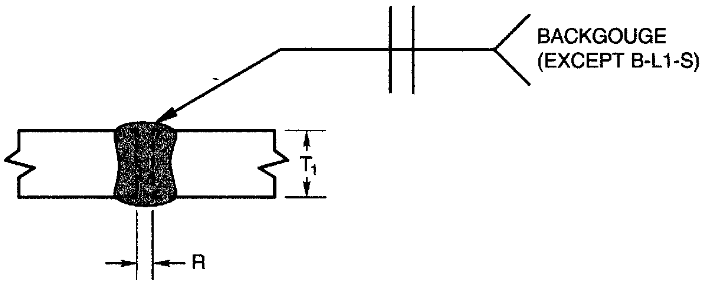
B-L1a



C-L1a

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
					Tolerances				
		T_1	T_2	Root Opening	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-L1a	$^1/_4$ max	—	$R = T_1$	$+^1/_{16}, -0$	$+^1/_4, -^1/_{16}$	All	—	5, 10
	C-L1a	$^1/_4$ max	U	$R = T_1$	$+^1/_{16}, -0$	$+^1/_4, -^1/_{16}$	All	—	5, 10
FCAW GMAW	B-L1a-GF	$^3/_8$ max	—	$R = T_1$	$+^1/_{16}, -0$	$+^1/_4, -^1/_{16}$	All	Not Required	1, 10

Square-groove weld (1)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
					Tolerances				
		T_1	T_2	Root Opening	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-L1b	$^1/_4$ max	—	$R = \frac{T_1}{2}$	$+^1/_{16}, -0$	$+^1/_{16}, -^1/_8$	All	—	4, 5, 10
GMAW FCAW	B-L1b-GF	$^3/_8$ max	—	$R = 0$ to $^1/_8$	$+^1/_{16}, -0$	$+^1/_{16}, -^1/_8$	All	Not Required	1, 4, 10
SAW	B-L1-S	$^3/_8$ max	—	$R = 0$	±0	$+^1/_{16}, -0$	F	—	10
SAW	B-L1a-S	$^5/_8$ max	—	$R = 0$	±0	$+^1/_{16}, -0$	F	—	4, 10

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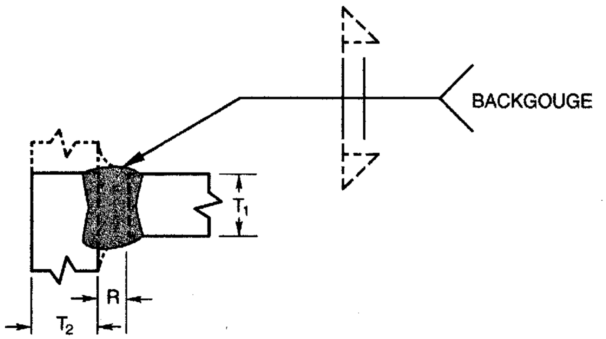
CJP

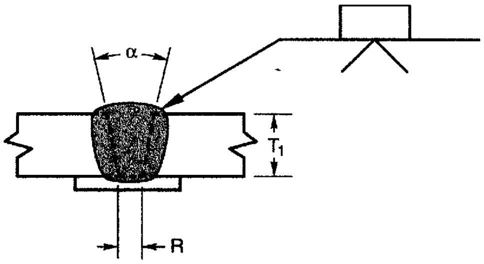
Table 8–2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Square-groove weld (1)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	TC-L1b	$^1/_4$ max	U	$R = \frac{T_1}{2}$	$+^1/_{16}, -0$	$+^1/_{16}, -^1/_8$	All	—	4, 5, 7
GMAW FCAW	TC-L1-GF	$^3/_8$ max	U	R = 0 to $^1/_8$	$+^1/_{16}, -0$	$+^1/_{16}, -^1/_8$	All	Not Required	1, 4, 7
SAW	TC-L1-S	$^3/_8$ max	U	R = 0	±0	$+^1/_{16}, -0$	F	—	4, 7

Single-V-groove weld (2)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
$R = +^1/_{16}, -0$	$+^1/_4, -^1/_{16}$
$\alpha = +10°, -0°$	$+10°, -5°$

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	B-U2a	U	—	$R = ^1/_4$	$\alpha = 45°$	All	—	5, 10
				$R = ^3/_8$	$\alpha = 30°$	F, V, OH	—	5, 10
				$R = ^1/_2$	$\alpha = 20°$	F, V, OH	—	5, 10
GMAW FCAW	B-U2a-GF	U	—	$R = ^3/_{16}$	$\alpha = 30°$	F, V, OH	Required	1, 10
				$R = ^3/_8$	$\alpha = 30°$	F, V, OH	Not req.	1, 10
				$R = ^1/_4$	$\alpha = 45°$	F, V, OH	Not req.	1, 10
SAW	B-L2a-S	2 max	—	$R = ^1/_4$	$\alpha = 30°$	F	—	10
SAW	B-U2-S	U	—	$R = ^5/_8$	$\alpha = 20°$	F	—	10

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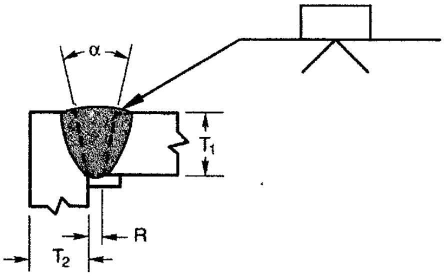
Table 8–2 (continued)

CJP

Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Single-V-groove weld (2)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/4, −1/16
α = +10°, −0°	+10°, −5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	C-U2a	U	U	R = 1/4	α = 45°	All	—	5, 10
				R = 3/8	α = 30°	F, V, OH	—	5, 10
				R = 1/2	α = 20°	F, V, OH	—	5, 10
GMAW FCAW	C-U2a-GF	U	U	R = 3/16	α = 30°	F, V, OH	Required	1
				R = 3/8	α = 30°	F, V, OH	Not req.	1, 10
				R = 1/4	α = 45°	F, V, OH	Not req.	1, 10
SAW	C-L2a-S	2 max	U	R = 1/4	α = 30°	F	—	10
SAW	C-U2-S	U	U	R = 5/8	α = 20°	F	—	10

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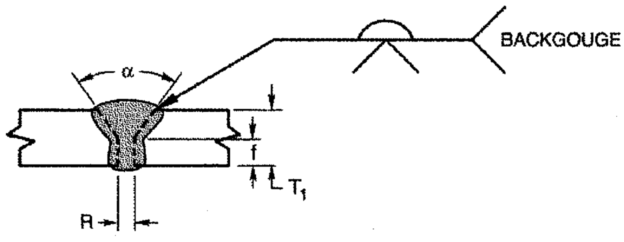
Table 8–2 (continued)

Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Single-V-groove weld (2)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.13.1)	Tolerances: As Fit-Up (see 3.13.1)			
SMAW	B-U2	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 + 10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	—	4, 5, 10
GMAW FCAW	B-U2-GF	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 + 10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	Not Required	1, 4, 10
SAW	B-L2c-S	Over 1/2 to 1	—	R = 0 f = 1/4 max α = 60°	R = ±0 f = +0, –f α = +10°, –0°	+1/16, –0 ± 1/16 +10°, –5°	F	—	4, 10
		Over 1 to 1 1/2	—	R = 0 f = 1/2 max α = 60°					
		Over 1 1/2 to 2	—	R = 0 f = 5/8 max α = 60°					

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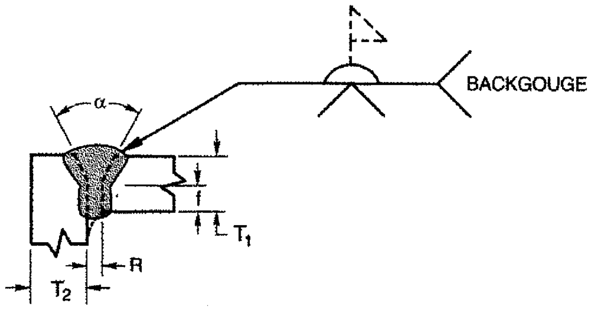
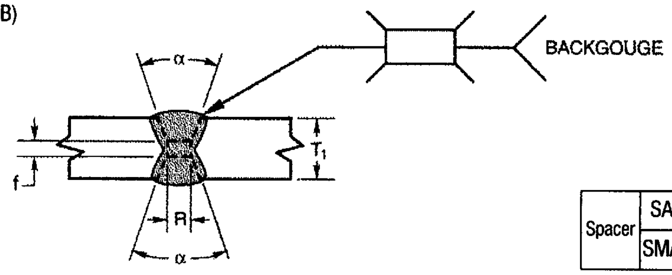
Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Single-V-groove weld (2)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances As Detailed (see 3.13.1)	Tolerances As Fit-Up (see 3.13.1)			
SMAW	C-U2	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 + 10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	—	4, 5, 7, 10
GMAW FCAW	C-U2-GF	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 60°	+1/16, –0 +1/16, –0 + 10°, –0°	+1/16, –1/8 Not Limited +10°, –5°	All	Not Required	1, 4, 7, 10
SAW	C-U2b-S	U	U	R = 0 to 1/8 f = 1/4 max α = 60°	±0 +0, –1/4 +10°, –0°	+1/16, –0 ±1/16 +10°, –5°	F	—	4, 7, 10

Double-V-groove weld (3)
Butt joint (B)



		Tolerances	
		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
		R = ±0	+1/4, –0
		f = ±0	+1/16, –0
		α = +10°, –0°	+10°, –5°
Spacer	SAW	±0	+1/16, –0
Spacer	SMAW	±0	1/8, –0

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Root Face	Groove Angle			
SMAW	B-U3a	U Spacer = 1/8 × R	—	R = 1/4	f = 0 to 1/8	α = 45°	All	—	4, 5, 8, 10
				R = 3/8	f = 0 to 1/8	α = 30°	F, V, OH	—	
				R = 1/2	f = 0 to 1/8	α = 20°	F, V, OH	—	
SAW	B-U3a-S	U Spacer = 1/4 × R	—	R = 5/8	f = 0 to 1/4	α = 20°	F	—	4, 8, 10

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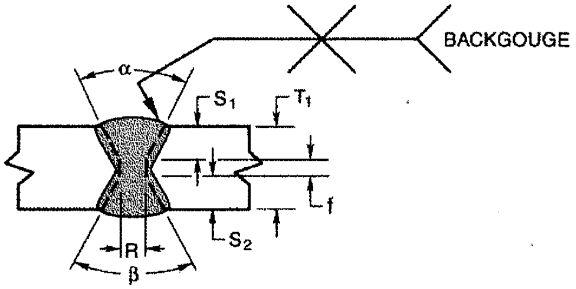
Table 8–2 (continued)
Prequalified Welded Joints
Complete-Joint-Penetration Groove Welds

Double-V-groove weld (3)
Butt joint (B)



For B-U3c-S only		
T_1		S_1
Over	to	
2	$2\frac{1}{2}$	$1\frac{3}{8}$
$2\frac{1}{2}$	3	$1\frac{3}{4}$
3	$3\frac{5}{8}$	$2\frac{1}{8}$
$3\frac{5}{8}$	4	$2\frac{3}{8}$
4	$4\frac{3}{4}$	$2\frac{3}{4}$
$4\frac{3}{4}$	$5\frac{1}{2}$	$3\frac{1}{4}$
$5\frac{1}{2}$	$6\frac{1}{4}$	$3\frac{3}{4}$
For $T_1 > 6\frac{1}{4}$ or $T_1 \leq 2$ $S_1 = \frac{2}{3}(T_1 - \frac{1}{4})$		

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
					Tolerances				
		T_1	T_2	Root Opening Root Face Groove Angle	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-U3b	U	—	$R = 0$ to $\frac{1}{8}$ $f = 0$ to $\frac{1}{8}$ $\alpha = \beta = 60°$	$+\frac{1}{16}, -0$ $+\frac{1}{16}, -0$ $+10°, -0°$	$+\frac{1}{16}, -\frac{1}{8}$ Not limited $+10°, -5°$	All	—	4, 5, 8, 10
GMAW FCAW	B-U3-GF						All	Not required	1, 4, 8, 10
SAW	B-U3c-S	U	—	$R = 0$ $f = \frac{1}{4}$ min $\alpha = \beta = 60°$	$+\frac{1}{16}, -0$ $+\frac{1}{4}, -0$ $+10°, -0°$	$+\frac{1}{16}, -0$ $+\frac{1}{4}, -0$ $+10°, -5°$	F	—	4, 8, 10
				To find S_1 see table above: $S_2 = T_1 - (S_1+f)$					

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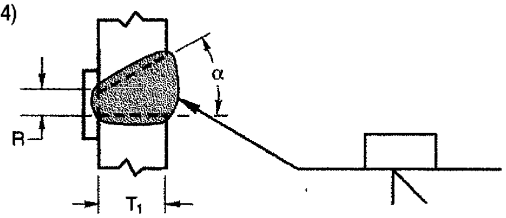
Table 8–2 (continued)

CJP

Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

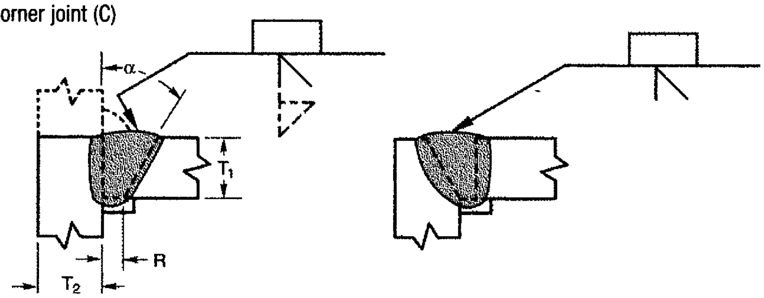
Single-bevel-groove weld (4)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/4, −1/16
α = +10°, −0°	+10°, −5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	B-U4a	U	—	R = 1/4	α = 45°	All	—	3, 5, 10
				R = 3/8	α = 30°	All	—	3, 5, 10
GMAW FCAW	B-U4a-GF	U	—	R = 3/16	α = 30°	All	Required	1, 3, 10
				R = 1/4	α = 45°	All	Not req.	1, 3, 10
				R = 3/8	α = 30°	F, H	Not req.	1, 3, 10
SAW	B-U4a-S	U	U	R = 3/8	α = 30°	F	—	3, 10
				R = 1/4	α = 45°			

Single-bevel-groove weld (4)
T-joint (T)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/4, −1/16
α = +10°, −0°	+10°, −5°

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation		Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle			
SMAW	TC-U4a	U	U	R = 1/4	α = 45°	All	—	5, 7, 10, 11
				R = 3/8	α = 30°	F, V, OH	—	5, 7, 10, 11
GMAW FCAW	TC-U4a-GF	U	U	R = 3/16	α = 30°	All	Required	1, 7, 10, 11
				R = 3/8	α = 30°	F	Not req.	1, 7, 10, 11
				R = 1/4	α = 45°	All	Not req.	1, 7, 10, 11
SAW	TC-U4a-S	U	U	R = 3/8	α = 30°	F	—	7, 10, 11
				R = 1/4	α = 45°			

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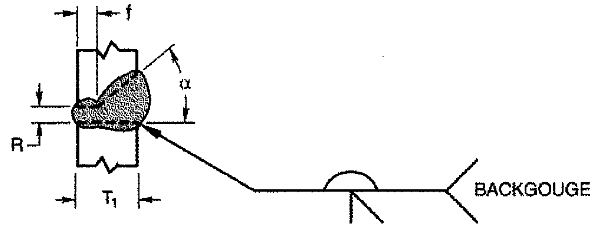
Table 8–2 (continued)

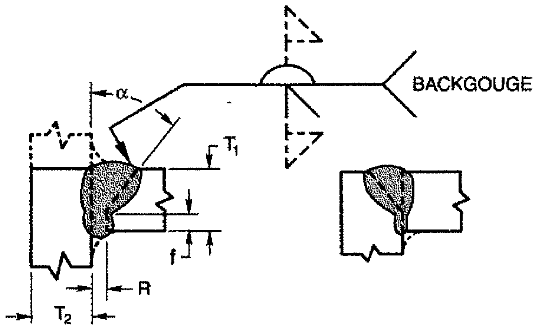
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Single-bevel-groove weld (4)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.13.1)	Tolerances: As Fit-Up (see 3.13.1)			
SMAW	B-U4b	U	—	R = 0 to 1/8 f = 0 to 1/8 α = 45°	+1/16, −0 +1/16, −0 + 10°, −0°	+1/16, −1/8 Not Limited +10°, −5°	All	—	3, 4, 5, 10
GMAW FCAW	B-U4b-GF	U	—				All	Not Required	1, 3, 4, 10
SAW	B-U4b-S	U	U	R = 0 f = 1/4 max α = 60°	±0 +0, −1/8 + 10°, −0°	+1/4, −0 ±1/16 10°, −5°	F	—	3, 4, 10

Single-bevel-groove weld (4)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances: As Detailed (see 3.13.1)	Tolerances: As Fit-Up (see 3.13.1)			
SMAW	TC-U4b	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 45°	+1/16, −0 +1/16, −0 + 10°, −0°	+1/16, −1/8 Not Limited +10°, −5°	All	—	4, 5, 7, 10, 11
GMAW FCAW	TC-U4b-GF	U	U				All	Not Required	1, 4, 7, 10, 11
SAW	TC-U4b-S	U	U	R = 0 f = 1/4 max α = 60°	±0 +0, −1/8 + 10°, −0°	+1/4, −0 ±1/16 10°, −5°	F	—	4, 7, 10, 11

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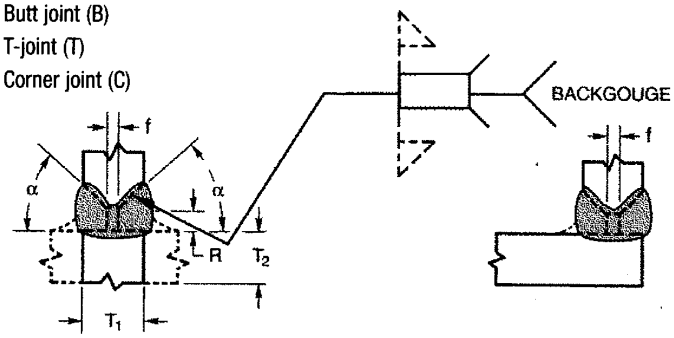
Table 8–2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

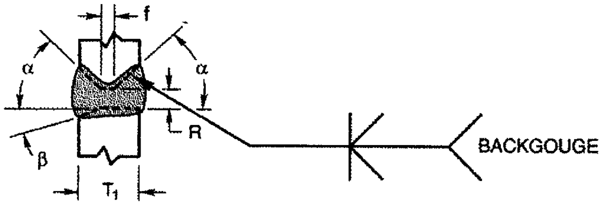
Double-bevel-groove weld (5)
Butt joint (B)
T-joint (T)
Corner joint (C)



	Tolerances	
	As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
	R = ±0	$+^1/_4, -0$
	$f = +^1/_{16}, -0$	$\pm^1/_{16}$
	$\alpha = +10^\circ, -0^\circ$	$+10^\circ, -5^\circ$
Spacer	$+^1/_{16}, -0$	$+^1/_8, -0$

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Root Face	Groove Angle			
SMAW	B-U5b	U Spacer = $^1/_8\times R$	U	$R = {}^1/_4$	$f = 0$ to $^1/_8$	$\alpha = 45^\circ$	All	—	3, 4, 5, 8, 10
	TC-U5a	U Spacer = $^1/_4\times R$	U	$R = {}^1/_4$	$f = 0$ to $^1/_8$	$\alpha = 45^\circ$	All	—	4, 5, 7, 8, 10, 11
				$R = {}^3/_8$	$f = 0$ to $^1/_8$	$\alpha = 30^\circ$	F, OH	—	4, 5, 7, 8, 10, 11

Double-bevel-groove weld
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances				
					As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	B-U5a	U	—	$R = 0$ to $^1/_8$ $f = 0$ to $^1/_8$ $\alpha = 45^\circ$ $\beta = 0^\circ$ to 15°	$+^1/_{16}, -0$ $+^1/_{16}, -0$ $\alpha + \beta\,^{+10^\circ}_{\;0^\circ}$	$+^1/_{16}, -^1/_8$ Not limited $\alpha + \beta\,^{+10^\circ}_{-5^\circ}$	All	—	3, 4, 5, 8, 10
GMAW FCAW	B-U5-GF	U	—	$R = 0$ to $^1/_8$ $f = 0$ to $^1/_8$ $\alpha = 45^\circ$ $\beta = 0^\circ$ to 15°	$+^1/_{16}, -0$ $+^1/_{16}, -0$ $\alpha + \beta =$ $+10^\circ, -0^\circ$	$+^1/_{16}, -^1/_8$ Not limited $\alpha + \beta =$ $+10^\circ, -5^\circ$	All	Not Required	1, 3, 4, 8, 10

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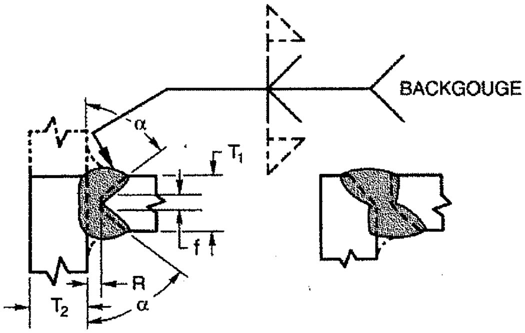
Table 8–2 (continued)

Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Double-bevel-groove weld (5)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Gas Shielding for FCAW	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)			
SMAW	TC-U5b	U	U	R = 0 to 1/8 f = 0 to 1/8 α = 45°	+1/16, −0 +1/16, −0 +10°, −0	+1/16, −1/8 Not limited +10°, −5°	All	—	4, 5, 7, 8, 10, 11
GMAW FCAW	TC-U5-GF	U	U				All	Not Required	1, 4, 7, 8, 10, 11
SAW	TC-U5-S	U	U	R = 0 f = 1/4 max α = 60°	± 0 +0, −3/16 +10°, −0°	+1/16, −0 ±1/16 +10°, −5°	F	—	4, 7, 8, 10, 11

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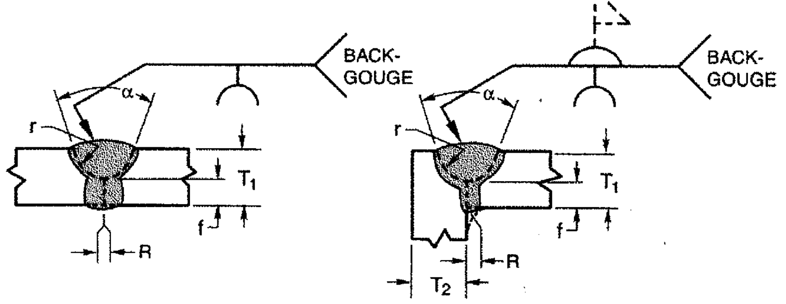
Table 8-2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Single-U-groove weld (6)
Butt joint (B)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, −0	+1/16, −1/8
α = +10°, −0°	+10°, −5°
f = ±1/16	Not Limited
r = +1/8, −0	+1/8, −0

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U6	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 1/4	All	—	4, 5, 10
				R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	F, OH	—	4, 5, 10
	C-U6	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 1/4	All	—	4, 5, 7, 10
				R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	F, OH	—	4, 5, 7, 10
GMAW FCAW	B-U6-GF	U	U	R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	All	Not req.	1, 4, 10
	C-U6-GF	U	U	R = 0 to 1/8	α = 20°	f = 1/8	r = 1/4	All	Not req.	1, 4, 7, 10

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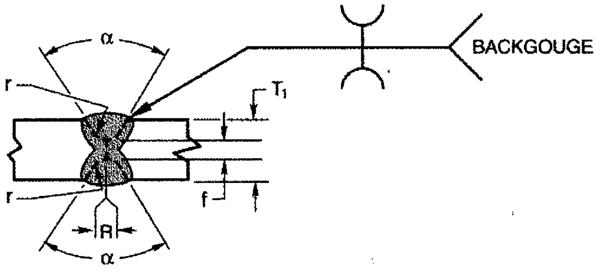
Table 8–2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Double-U-groove weld (7)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
For B-U7 and B-U7-GF	
$R = +{}^1/_{16}, -0$	${}^1/_{16}, -{}^1/_8$
$\alpha = +10°, -0°$	$+10°, -5°$
$f = \pm{}^1/_{16}, -0$	Not Limited
$r = +{}^1/_4, -0$	$\pm{}^1/_{16}$
For B-U7-S	
$R = \pm 0$	$+{}^1/_{16}, -0$
$f = +0, +{}^1/_4$	$\pm{}^1/_{16}$

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U7	U	—	$R = 0$ to ${}^1/_8$	$\alpha = 45°$	$f = {}^1/_8$	$r = {}^1/_4$	All	—	4, 5, 8, 10
				$R = 0$ to ${}^1/_8$	$\alpha = 20°$	$f = {}^1/_8$	$r = {}^1/_4$	F, OH	—	4, 5, 8, 10
GMAW FCAW	B-U7-GF	U	—	$R = 0$ to ${}^1/_8$	$\alpha = 20°$	$f = {}^1/_8$	$r = {}^1/_4$	All	Not req.	1, 4, 10, 8
SAW	B-U7-S	U	—	$R = 0$	$\alpha = 20°$	$f = {}^1/_4$ max	$r = {}^1/_4$	F	—	4, 8, 10

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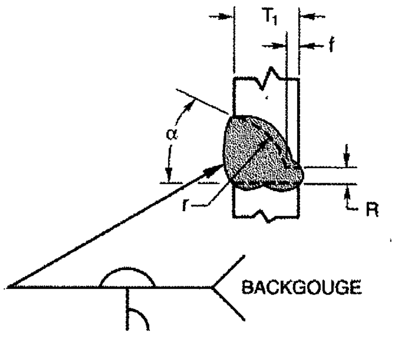
Table 8–2 (continued)

Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Single-J-groove weld (8)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
B-U8 and B-U8-GF	
R = +1/16, –0	+1/16, –1/8
α = +10°, –0°	+10°, –5°
f = +1/8, –0	Not Limited
r = +1/4, –0	±1/16
B-U8-S	
R = ±0	+1/4, –0
α = +10°, –0°	+10°, –5°
f = +0, –1/8	±1/16
r = +1/4, –0	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U8	U	—	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	3, 4, 5, 10
GMAW FCAW	B-U8-GF	U	—	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not req.	1, 3, 4, 10
SAW	B-U8-S	U	U	R = 0	α = 45°	f = 1/4 max	r = 3/8	F	—	3, 4, 10

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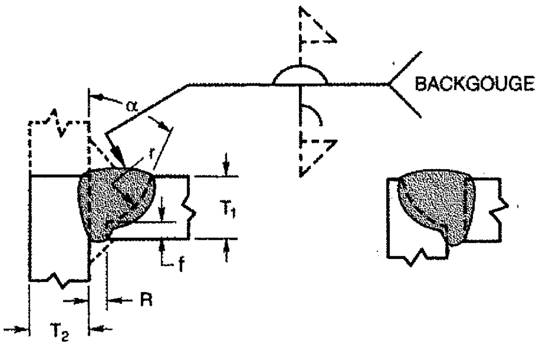
Table 8–2 (continued)
Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

Single-J-groove weld (8)
T-joint (T)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
TC-U8a and TC-U8a-GF	
R = +1/16, −0	1/16, −1/8
α = +10°, −0°	+10°, −5°
f = +1/16, −0	Not Limited
r = +1/4, −0	±1/16
TC-U8a-S	
R = ±0	+1/4, −0
α = +10°, −0°	+10°, −5°
f = +0, −1/8	±1/16
r = +1/4, −0	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	TC-U8a	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	4, 5, 7, 10, 11
				R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	F, OH	—	4, 5, 7, 10, 11
GMAW FCAW	TC-U8a-GF	U	U	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not req.	1, 4, 7, 10, 11
SAW	TC-U8a-S	U	U	R = 0	α = 45°	f = 1/4 max	r = 3/8	F	—	4, 7, 10, 11

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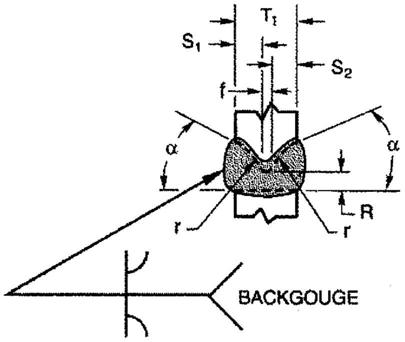
Table 8–2 (continued)

Prequalified Welded Joints Complete-Joint-Penetration Groove Welds

CJP

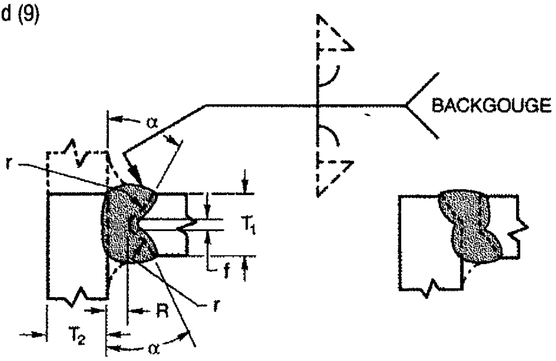
Double-J-groove weld (9)
Butt joint (B)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, –0	+1/16, –1/8
α = +10°, –0°	+10°, –5°
f = +1/16, –0	Not Limited
r = +1/8, –0	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	B-U9	U	—	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	3, 4, 5, 8, 10
GMAW FCAW	B-U9-GF	U	—	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not req.	1, 3, 4, 8, 10

Double-J-groove weld (9)
T-joint (T)
Corner joint (C)



Tolerances	
As Detailed (see 3.13.1)	As Fit-Up (see 3.13.1)
R = +1/16, –0	+1/16, –1/8
α = +10°, –0°	+10°, –5°
f = +1/16, –0	Not Limited
r = 1/8, –0	±1/16

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation				Allowed Welding Positions	Gas Shielding for FCAW	Notes
		T_1	T_2	Root Opening	Groove Angle	Root Face	Bevel Radius			
SMAW	TC-U9a	U	U	R = 0 to 1/8	α = 45°	f = 1/8	r = 3/8	All	—	4, 5, 7, 8, 10, 11
				R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	F, OH	—	4, 5, 7, 8, 11
GMAW FCAW	TC-U9a-GF	U	U	R = 0 to 1/8	α = 30°	f = 1/8	r = 3/8	All	Not req.	1, 4, 7, 8, 10, 11

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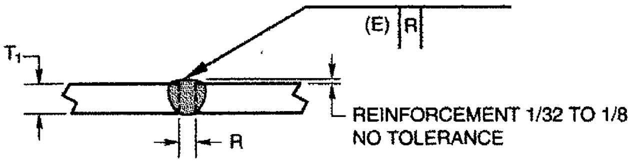
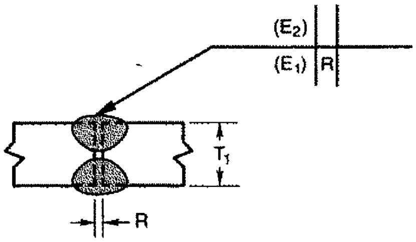
Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Square-groove weld (1)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Weld Size (E)	Notes
					Tolerances				
		T_1	T_2	Root Opening	As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P1a	1/8	—	R = 0 to 1/16	+1/16, –0	±1/16	All	$T_1 - 1/32$	2, 5
	B-P1c	1/4 max	—	$R = \frac{T_1}{2}$ min	+1/16, –0	±1/16	All	$\frac{T_1}{2}$	2, 5

Square-groove weld (1)
Butt joint (B)



$E_1 + E_2$ must not exceed $\frac{3T_1}{4}$

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
					Tolerances				
		T_1	T_2	Root Opening	As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P1b	1/4 max	—	$R = \frac{T_1}{2}$	+1/16, –0	±1/16	All	$\frac{3T_1}{4}$	5

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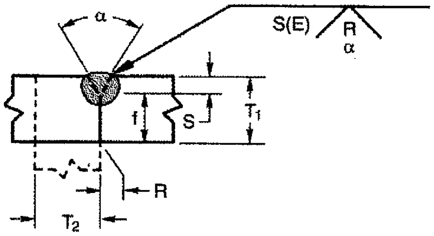
Table 8–2 (continued)
Prequalified Welded Joints Partial-Joint-Penetration Groove Welds

Single-V-groove weld (2)
Butt joint (B)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances As Detailed (see 3.12.3)	Tolerances As Fit-Up (see 3.12.3)			
SMAW	BC-P2	1/4 min	U	R = 0 f = 1/32 min α = 60°	−0, +1/16 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	All	S	2, 5, 6, 10
GMAW FCAW	BC-P2-GF	1/4 min	U	R = 0 f = 1/8 min α = 60°	−0, +1/16 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	All	S	1, 2, 6, 10
SAW	BC-P2-S	7/16 min	U	R = 0 f = 1/4 min α = 60°	±0 +U, −0 +10°, −0°	+1/16, −0 ±1/16 + 10°, −5°	F	S	2, 6, 10

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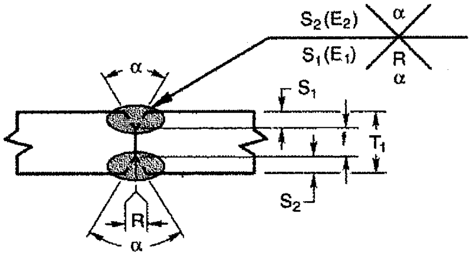
PJP

Table 8–2 (continued)

Prequalified Welded Joints Partial-Joint-Penetration Groove Welds

Double-V-groove weld (3)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
				Root Opening Root Face Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P3	1/2 min	—	R = 0 f = 1/8 min α = 60°	+1/16, –0 +U, –0 +10°, –0°	+1/8, –1/16 ±1/16 + 10°, –5°	All	$S_1 + S_2$	5, 6, 9, 10
GMAW FCAW	B-P3-GF	1/2 min	—	R = 0 f = 1/8 min α = 60°	+1/16, –0 +U, –0 +10°, –0°	+1/8, –1/16 ±1/16 + 10°, –5°	All	$S_1 + S_2$	1, 6, 9, 10
SAW	B-P3-S	3/4 min	—	R = 0 f = 1/4 min α = 60°	±0 +U, –0 +10°, –0°	+1/16, –0 ±1/16 + 10°, –5°	F	$S_1 + S_2$	6, 9, 10

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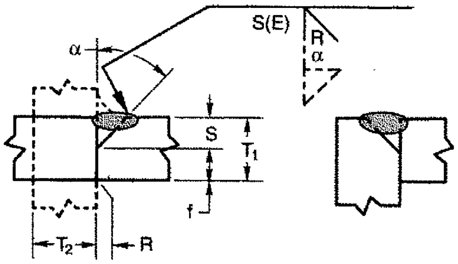
Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Single-bevel-groove weld (4)
Butt joint (B)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances As Detailed (see 3.12.3)	Tolerances As Fit-Up (see 3.12.3)			
SMAW	BTC-P4	U	U	R = 0 f = 1/8 min α = 45°	+1/16, –0 +U, –0 +10°, –0°	+1/8, –1/16 ±1/16 + 10°, –5°	All	S–1/8	2, 5, 6, 7, 10, 11
GMAW FCAW	BTC-P4-GF	1/4 min	U	R = 0 f = 1/8 min α = 45°	+1/16, –0 +U, –0 +10°, –0°	+1/8, –1/16 ±1/16 + 10°, –5°	F, H V, OH	S S–1/8	1, 2, 6, 7, 10, 11
SAW	TC-P4-S	7/16 min	U	R = 0 f = 1/4 min α = 60°	±0 +U, –0 +10°, –0°	+1/16, –0 ±1/16 + 10°, –5°	F	S	2, 6, 7, 10, 11

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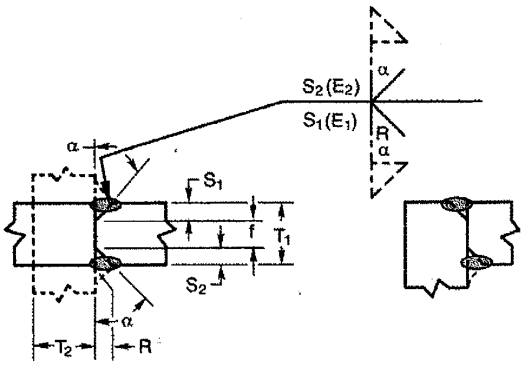
Table 8–2 (continued)

Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Double-bevel-groove weld (5)
Butt joint (B)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
		T_1	T_2	Root Opening Root Face Groove Angle	Tolerances As Detailed (see 3.12.3)	Tolerances As Fit-Up (see 3.12.3)			
SMAW	BTC-P5	5/16 min	U	R = 0 f = 1/8 min α = 45°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	All	$S_1 + S_2$ −1/4	5, 6, 7, 9, 10, 11
GMAW FCAW	BTC-P5-GF	1/2 min	U	R = 0 f = 1/8 min α = 45°	+1/16, −0 +U, −0 +10°, −0°	+1/8, −1/16 ±1/16 + 10°, −5°	F, H	$S_1 + S_2$	1, 6, 7, 9, 10, 11
							V, OH	$S_1 + S_2$ −1/4	
SAW	TC-P5-S	3/4 min	U	R = 0 f = 1/4 min α = 60°	±0 +U, −0 +10°, −0°	+1/16, −0 ±1/16 + 10°, −5°	F	$S_1 + S_2$	6, 7, 9, 10, 11

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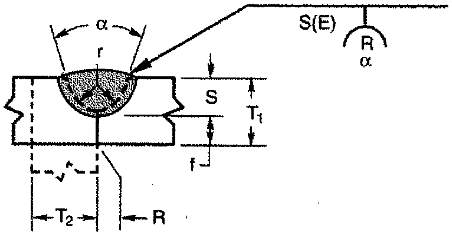
Table 8–2 (continued)

Prequalified Welded Joints Partial-Joint-Penetration Groove Welds

Single-U-groove weld (6)
Butt joint (B)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Bevel Radius Groove Angle	Tolerances				
					As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	BC-P6	1/4 min	U	R = 0 f = 1/32 min r = 1/4 α = 45°	+1/16, –0 +U, –0 +1/4, –0 +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 + 10°, –5°	All	S	2, 5, 6, 10
GMAW FCAW	BC-P6-GF	1/4 min	U	R = 0 f = 1/8 min r = 1/4 α = 20°	+1/16, –0 +U, –0 +1/4, –0 +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 + 10°, –5°	All	S	1, 2, 6, 10
SAW	BC-P6-S	7/16 min	U	R = 0 f = 1/4 min r = 1/4 α = 20°	±0 +U, –0 +1/4, –0 +10°, –0°	+1/16, –0° ±1/16 ±1/16 + 10°, –5°	F	S	2, 6, 10

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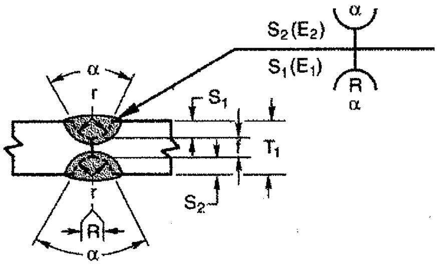
Table 8–2 (continued)

Prequalified Welded Joints Partial-Joint-Penetration Groove Welds

Double-U-groove weld (7)
Butt joint (B)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
				Root Opening Root Face Bevel Radius Groove Angle	Tolerances				
		T_1	T_2		As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P7	1/2 min	—	R = 0 f = 1/8 min r = 1/4 α = 45°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 + 10°, −5°	All	$S_1 + S_2$	5, 6, 9, 10
GMAW FCAW	B-P7-GF	1/2 min	—	R = 0 f = 1/8 min r = 1/4 α = 20°	+1/16, −0 +U, −0 +1/4, −0 +10°, −0°	+1/8, −1/16 ±1/16 ±1/16 + 10°, −5°	All	$S_1 + S_2$	1, 6, 9, 10
SAW	B-P7-S	3/4 min	—	R = 0 f = 1/4 min r = 1/4 α = 20°	±0 +U, −0 +1/4, −0 +10°, −0°	+1/16, −0° ±1/16 ±1/16 + 10°, −5°	F	$S_1 + S_2$	6, 9,10

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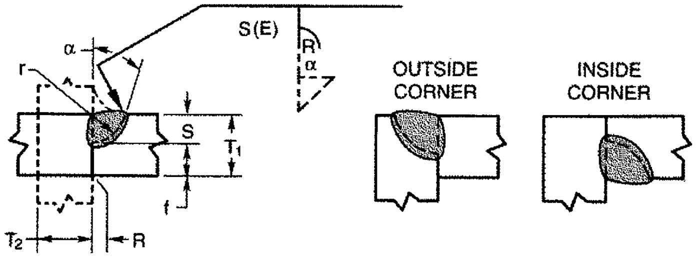
Table 8–2 (continued)
Prequalified Welded Joints Partial-Joint-Penetration Groove Welds

Single-J-groove weld (8)
Butt joint (B)
T-joint (T)
Corner joint (C)



*α_{oc} = Outside corner groove angle.
**α_{ic} = Inside corner groove angle.

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
		T_1	T_2	Root Opening Root Face Bevel Radius Groove Angle	Tolerances				
					As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	B-P8	1/4 min	U	R = 0 f = 1/8 min r = 3/8 α = 30°	+1/16, –0 +U, –0 +1/4, –0 +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5°	All	S	5, 6, 7, 10, 11
	TC-P8	1/4 min	U	R = 0 f = 1/8 min r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	+1/16, –0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5° +10°, –5°	All	S	5, 6, 7, 10, 11
GMAW FCAW	B-P8-GF	1/4 min	U	R = 0 f = 1/8 min r = 3/8 α = 30°	+1/16, –0 +U, –0 +1/4, –0 +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5°	All	S	1, 6, 7, 10, 11
	TC-P8-GF	1/4 min	U	R = 0 f = 1/8 min r = 3/8 α_{oc} = 30°* α_{ic} = 45°**	+1/16, –0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/8, –1/16 ±1/16 ±1/16 +10°, –5° +10°, –5°	All	S	1, 6, 7, 10, 11
SAW	B-P8-S	7/16 min	U	R = 0 f = 1/4 min r = 1/2 α = 20°	±0 +U, –0 +1/4, –0 +10°, –0°	+1/16, –0 ±1/16 ±1/16 +10°, –5°	F	S	6, 7, 10, 11
	TC-P8-S	7/16 min	U	R = 0 f = 1/4 min r = 1/2 α_{oc} = 20°* α_{ic} = 45°**	±0 +U, –0 +1/4, –0 +10°, –0° +10°, –0°	+1/16, –0 ±1/16 ±1/16 +10°, –5° +10°, –5°	F	S	6, 7, 10, 11

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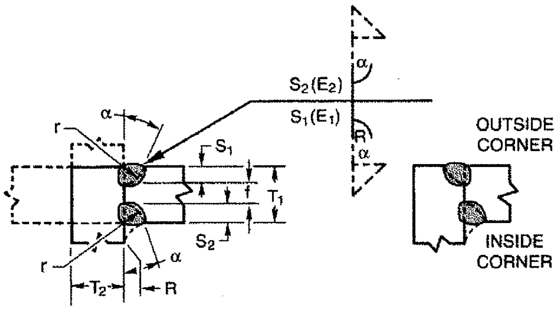
PJP

Table 8–2 (continued)
Prequalified Welded Joints
Partial-Joint-Penetration Groove Welds

Double-J-groove weld (9)
Butt joint (B)
T-joint (T)
Corner joint (C)



$^*\alpha_{oc}$ = Outside corner groove angle.
$^{**}\alpha_{ic}$ = Inside corner groove angle.

Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)		Groove Preparation			Allowed Welding Positions	Total Weld Size ($E_1 + E_2$)	Notes
		T_1	T_2	Root Opening Root Face Bevel Radius Groove Angle	Tolerances As Detailed (see 3.12.3)	Tolerances As Fit-Up (see 3.12.3)			
SMAW	B-P9	$^1/_2$ min	U	R = 0 f = $^1/_8$ min r = $^3/_8$ $\alpha = 30°$	$+^1/_{16}, -0$ +U, −0 $+^1/_4, -0$ +10°, −0°	$+^1/_8, -^1/_{16}$ $\pm^1/_{16}$ $\pm^1/_{16}$ +10°, −5°	All	$S_1 + S_2$	5, 6, 7, 9, 10, 11
	TC-P9	$^1/_2$ min	U	R = 0 f = $^1/_8$ min r = $^3/_8$ $\alpha_{oc} = 30°^*$ $\alpha_{ic} = 45°^{**}$	$+^1/_{16}, -0$ +U, −0 $+^1/_4, -0$ +10°, −0° +10°, −0°	$+^1/_8, -^1/_{16}$ $\pm^1/_{16}$ $\pm^1/_{16}$ +10°, −5° +10°, −5°	All	$S_1 + S_2$	5, 6, 7, 9, 10, 11
GMAW FCAW	B-P9-GF	$^1/_2$ min	U	R = 0 f = $^1/_8$ min r = $^3/_8$ $\alpha = 30°$	$+^1/_{16}, -0$ +U, −0 $+^1/_4, -0$ +10°, −0°	$+^1/_8, -^1/_{16}$ $\pm^1/_{16}$ $\pm^1/_{16}$ +10°, −5°	All	$S_1 + S_2$	1, 6, 7, 9, 10, 11
	TC-P9-GF	$^1/_2$ min	U	R = 0 f = $^1/_8$ min r = $^3/_8$ $\alpha_{oc} = 30°^*$ $\alpha_{ic} = 45°^{***}$	±0 +U, −0 $+^1/_4, -0$ +10°, −0° +10°, −0°	$+^1/_{16}, -0$ $\pm^1/_{16}$ $\pm^1/_{16}$ +10°, −5° +10°, −5°	All	$S_1 + S_2$	1, 6, 7, 9, 10, 11
SAW	B-P9-S	$^3/_4$ min	U	R = 0 f = $^1/_4$ min r = $^1/_2$ $\alpha = 20°$	±0 +U, −0 $+^1/_4, -0$ +10°, −0°	$+^1/_{16}, -0$ $\pm^1/_{16}$ $\pm^1/_{16}$ +10°, −5°	F	$S_1 + S_2$	6, 7, 9, 10, 11
	TC-P9-S	$^3/_4$ min	U	R = 0 f = $^1/_4$ min r = $^1/_2$ $\alpha_{oc} = 20°^*$ $\alpha_{ic} = 45°^{***}$	±0 +U, −0 $+^1/_4, -0$ +10°, −0° +10°, −0°	$+^1/_{16}, -0$ $\pm^1/_{16}$ $\pm^1/_{16}$ +10°, −5° +10°, −5°	F	$S_1 + S_2$	6, 7, 9, 10, 11

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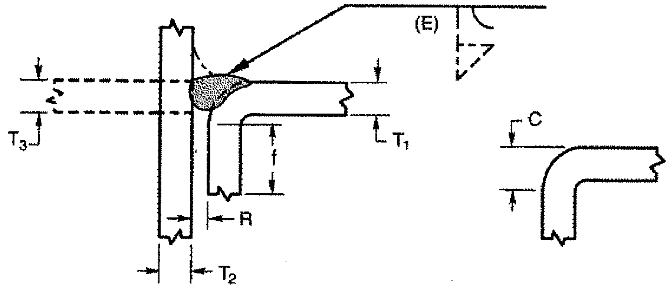
FLARE

Table 8–2 (continued)

Prequalified Welded Joints Flare-Bevel Groove Welds

Flare-bevel-groove weld (10)
Butt joint (B)
T-joint (T)
Corner joint (C)



Welding Process	Joint Designation	Base Metal Thickness (U = unlimited)			Groove Preparation			Allowed Welding Positions	Total Weld Size (E)	Notes
					Root Opening Root Face Bend Radius*	Tolerances				
		T_1	T_2	T_3		As Detailed (see 3.12.3)	As Fit-Up (see 3.12.3)			
SMAW	BTC-P10	3/16 min	U	T_1 min	R = 0 f = 3/16 min $C = \frac{3T_1}{2}$ min	+1/16, −0 +U, −0 +U, −0	+1/8, −1/16 +U, −1/16 +U, −0	All	$\frac{5T_1}{8}$	5, 7, 10, 12
GMAW FCAW	BTC-P10-GF	3/16 min	U	T_1 min	R = 0 f = 3/16 min $C = \frac{3T_1}{2}$ min	+1/16, −0 +U, −0 +U, −0	+1/8, −1/16 +U, −1/16 +U, −0	All	$\frac{5T_1}{8}$	1, 7, 10, 12
SAW	T-P10-S	1/2 min	1/2 min	N/A	R = 0 f = 1/2 min $C = \frac{3T_1}{2}$ min	±0 +U, −0 +U, −0	+1/16, −0° +U, −1/16 +U, −0	F	$\frac{5T_1}{8}$	7, 10, 12

* For cold formed (A500) rectangular tubes, C dimension is not limited. See the following:
Effective Weld Size of Flare-Bevel-Groove Welded Joints. Tests have been performed on cold formed ASTM A 500 material exhibiting a "C" dimension as small as T1 with a nominal radius of 2t. As the radius increases, the "C" dimension also increases. The corner curvature may not be a quadrant of a circle tangent to the sides. The corner dimension, "C," may be less than the radius of the corner.

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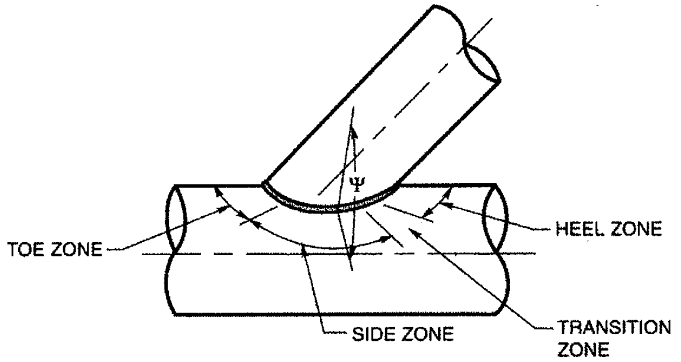
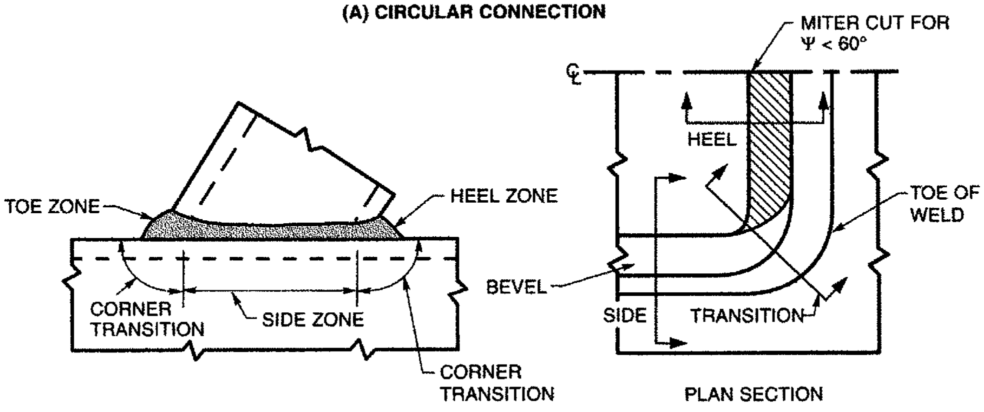
Table 8–2 (continued)

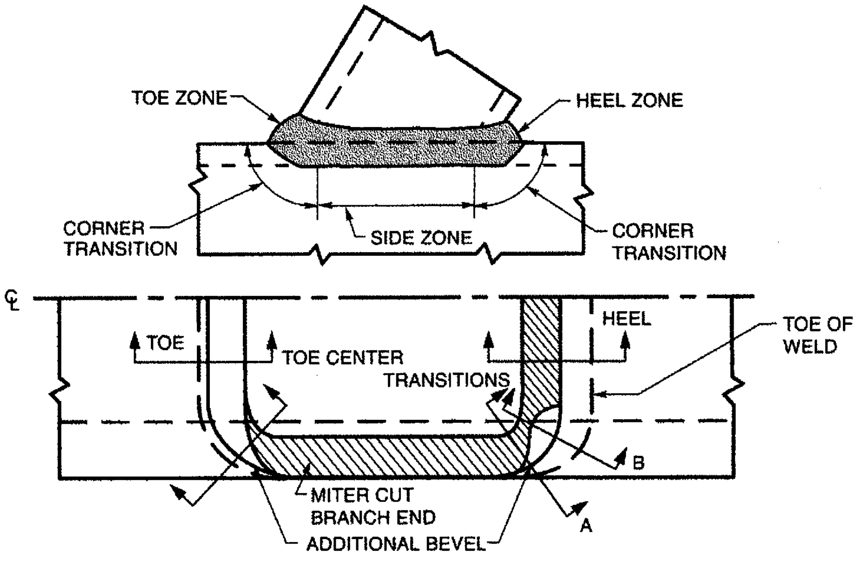
Prequalified Welded Joints PJP T-, Y-, and K-Tubular Connections



(A) CIRCULAR CONNECTION



(B) STEPPED BOX CONNECTION



(C) MATCHED BOX CONNECTION

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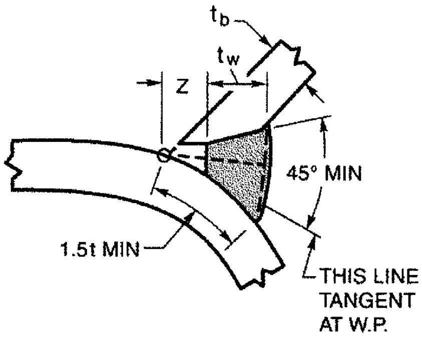
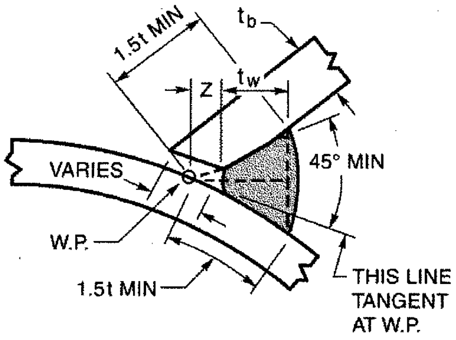
TUBE

Table 8–2 (continued)

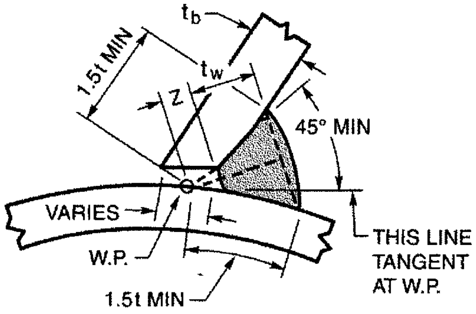
Prequalified Welded Joints PJP T-, Y-, and K-Tubular Connections



TRANSITION A

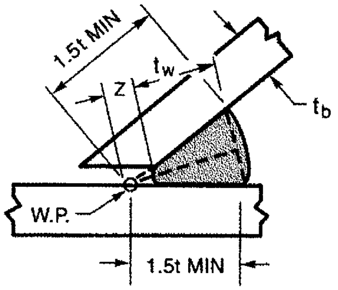


TRANSITION B



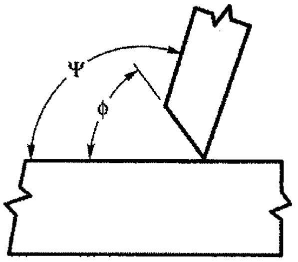
Ψ = 75°–60°

TRANSITION OR HEEL



Ψ = 60°–30°

HEEL



SKETCH FOR ANGULAR DEFINITION

150° ≥ Ψ ≥ 30°
90° > φ ≥ 30°

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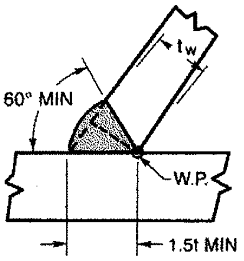
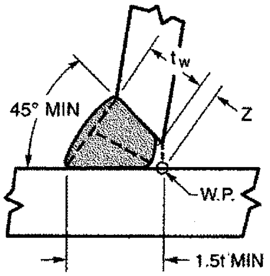
TUBE

Table 8–2 (continued)
Prequalified Welded Joints PJP T-, Y-, and K-Tubular Connections



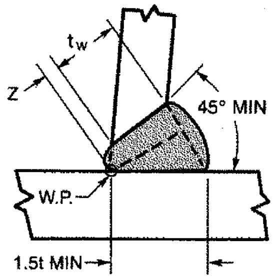
Ψ = 150°–105°

TOE



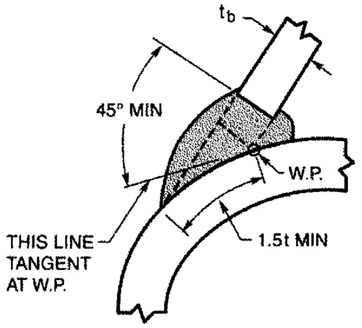
Ψ = 105°–90°

TOE OR HEEL

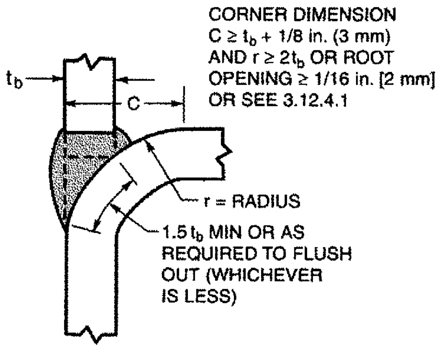


Ψ = 90°–75°

SIDE OR HEEL



TOE CORNER



SIDE MATCHED

General Notes:

- t = thickness of thinner section.
- Bevel to feather edge except in transition and heel zones.
- Root opening: 0 to 3/16 in. [5 mm].
- Not prequalified for under 30°.
- Weld size (effective throat) $t_w \geq t$; Z Loss Dimensions shown in Table 2.8.
- Calculations per 2.24.1.3 shall be done for leg length less than 1.5t, as shown.
- For Box Section, joint preparation for corner transitions shall provide a smooth transition from one detail to another. Welding shall be carried continuously around corners, with corners fully built up and all weld starts and stops within flat faces.
- See Annex B for definition of local dihedral angle, Ψ.
- W.P. = work point.

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Table 8–3
Electrode Strength Coefficient, C_1

Electrode	F_{EXX} (ksi)	C1
E60	60	0.857
E70	70	1.00
E80	80	1.03
E90	90	1.16
E100	100	1.21
E110	110	1.34

Table 8-4

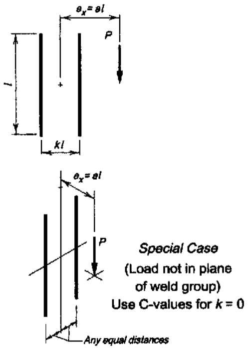
Coefficients *C* for Eccentrically Loaded Weld Groups

Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



Special Case
(Load not in plane of weld group)
Use C-values for k = 0

	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71	3.71
0.100	3.72	3.73	3.72	3.71	3.70	3.69	3.67	3.65	3.63	3.61	3.60	3.56	3.52	3.48	3.45	3.71
0.150	3.67	3.66	3.65	3.64	3.62	3.60	3.58	3.56	3.54	3.52	3.50	3.47	3.43	3.40	3.37	3.34
0.200	3.51	3.51	3.50	3.49	3.47	3.46	3.44	3.43	3.41	3.40	3.38	3.36	3.33	3.30	3.28	3.25
0.250	3.31	3.31	3.31	3.30	3.29	3.28	3.28	3.27	3.26	3.26	3.25	3.23	3.22	3.20	3.18	3.17
0.300	3.09	3.09	3.09	3.10	3.10	3.10	3.11	3.11	3.11	3.11	3.11	3.11	3.11	3.10	3.09	3.08
0.400	2.66	2.66	2.68	2.70	2.73	2.75	2.78	2.80	2.82	2.83	2.85	2.87	2.89	2.90	2.90	2.90
0.500	2.29	2.30	2.32	2.35	2.40	2.44	2.48	2.52	2.55	2.58	2.61	2.65	2.68	2.71	2.73	2.74
0.600	2.00	2.00	2.03	2.07	2.12	2.18	2.23	2.28	2.32	2.36	2.39	2.45	2.50	2.54	2.57	2.59
0.700	1.76	1.76	1.79	1.84	1.90	1.96	2.02	2.07	2.12	2.16	2.21	2.28	2.33	2.38	2.42	2.45
0.800	1.56	1.57	1.60	1.65	1.71	1.77	1.84	1.90	1.95	2.00	2.04	2.12	2.19	2.24	2.29	2.32
0.900	1.41	1.41	1.44	1.49	1.56	1.62	1.69	1.75	1.80	1.85	1.90	1.98	2.05	2.11	2.16	2.21
1.00	1.28	1.28	1.31	1.37	1.43	1.49	1.56	1.62	1.67	1.73	1.77	1.86	1.94	2.00	2.05	2.10
1.20	1.07	1.08	1.11	1.16	1.22	1.28	1.35	1.41	1.46	1.51	1.57	1.66	1.73	1.80	1.86	1.91
1.40	0.927	0.935	0.965	1.01	1.07	1.13	1.19	1.24	1.30	1.35	1.40	1.49	1.57	1.64	1.70	1.76
1.60	0.815	0.821	0.851	0.893	0.944	1.00	1.06	1.11	1.16	1.21	1.26	1.35	1.43	1.50	1.57	1.62
1.80	0.725	0.733	0.760	0.800	0.847	0.899	0.952	1.00	1.05	1.10	1.15	1.24	1.32	1.39	1.45	1.51
2.00	0.655	0.661	0.687	0.723	0.768	0.816	0.867	0.916	0.964	1.01	1.06	1.14	1.22	1.29	1.35	1.41
2.20	0.596	0.603	0.627	0.660	0.701	0.747	0.795	0.841	0.887	0.932	0.975	1.06	1.13	1.20	1.26	1.32
2.40	0.547	0.553	0.575	0.607	0.645	0.688	0.733	0.777	0.821	0.864	0.905	0.984	1.06	1.12	1.18	1.24
2.60	0.505	0.512	0.532	0.561	0.597	0.637	0.680	0.723	0.764	0.805	0.845	0.921	0.991	1.05	1.11	1.17
2.80	0.469	0.476	0.495	0.523	0.556	0.595	0.635	0.675	0.715	0.753	0.792	0.865	0.932	0.995	1.05	1.11
3.00	0.439	0.444	0.463	0.488	0.520	0.556	0.595	0.632	0.671	0.708	0.745	0.815	0.881	0.941	0.997	1.05

Table 8–4 (continued)

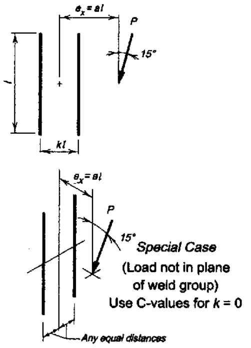
Coefficients *C* for Eccentrically Loaded Weld Groups

Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



a	k = 0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84
0.100	3.79	3.79	3.78	3.77	3.77	3.76	3.75	3.74	3.73	3.72	3.71	3.69	3.67	3.65	3.64	3.62
0.150	3.68	3.68	3.67	3.66	3.65	3.64	3.63	3.62	3.61	3.61	3.60	3.58	3.57	3.55	3.54	3.53
0.200	3.51	3.50	3.50	3.49	3.49	3.49	3.49	3.48	3.48	3.47	3.47	3.46	3.46	3.45	3.44	3.44
0.250	3.31	3.31	3.31	3.31	3.31	3.31	3.32	3.32	3.33	3.33	3.33	3.34	3.34	3.34	3.34	3.34
0.300	3.09	3.09	3.10	3.11	3.12	3.14	3.15	3.16	3.17	3.18	3.19	3.21	3.23	3.23	3.24	3.25
0.400	2.67	2.67	2.69	2.71	2.75	2.79	2.82	2.85	2.88	2.90	2.93	2.96	3.00	3.02	3.04	3.06
0.500	2.32	2.32	2.34	2.38	2.42	2.47	2.52	2.57	2.61	2.65	2.68	2.74	2.79	2.83	2.86	2.89
0.600	2.02	2.03	2.06	2.10	2.15	2.21	2.27	2.33	2.38	2.42	2.46	2.54	2.60	2.65	2.69	2.73
0.700	1.79	1.79	1.82	1.87	1.93	1.99	2.06	2.12	2.17	2.23	2.27	2.36	2.43	2.49	2.54	2.58
0.800	1.59	1.60	1.63	1.68	1.74	1.81	1.88	1.94	2.00	2.06	2.11	2.20	2.27	2.34	2.40	2.45
0.900	1.44	1.44	1.48	1.53	1.59	1.66	1.73	1.79	1.85	1.91	1.96	2.05	2.14	2.21	2.27	2.32
1.00	1.31	1.31	1.35	1.40	1.46	1.53	1.59	1.66	1.72	1.78	1.83	1.93	2.01	2.09	2.15	2.21
1.20	1.10	1.11	1.14	1.19	1.25	1.32	1.38	1.45	1.51	1.56	1.62	1.72	1.81	1.88	1.95	2.01
1.40	0.952	0.959	0.991	1.04	1.09	1.15	1.22	1.28	1.34	1.39	1.45	1.54	1.63	1.71	1.78	1.85
1.60	0.837	0.844	0.875	0.917	0.969	1.03	1.09	1.15	1.20	1.25	1.31	1.40	1.49	1.57	1.64	1.70
1.80	0.747	0.755	0.783	0.823	0.871	0.924	0.980	1.04	1.09	1.14	1.19	1.28	1.37	1.45	1.52	1.58
2.00	0.675	0.681	0.707	0.744	0.789	0.840	0.892	0.944	0.995	1.04	1.09	1.18	1.27	1.34	1.41	1.47
2.20	0.615	0.621	0.645	0.680	0.721	0.769	0.817	0.867	0.916	0.963	1.01	1.10	1.18	1.25	1.32	1.38
2.40	0.564	0.571	0.593	0.625	0.665	0.709	0.755	0.801	0.848	0.893	0.937	1.02	1.10	1.17	1.24	1.30
2.60	0.521	0.528	0.549	0.579	0.616	0.657	0.701	0.745	0.788	0.832	0.875	0.955	1.03	1.10	1.16	1.22
2.80	0.484	0.491	0.511	0.539	0.573	0.613	0.655	0.696	0.737	0.779	0.819	0.896	0.969	1.04	1.10	1.16
3.00	0.452	0.459	0.477	0.504	0.537	0.573	0.613	0.652	0.692	0.732	0.771	0.845	0.915	0.981	1.04	1.10

Table 8–4 (continued)

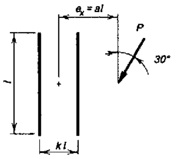
Coefficients *C* for Eccentrically Loaded Weld Groups

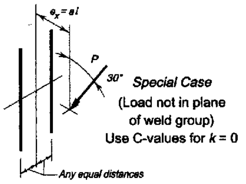
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)





Special Case
(Load not in plane of weld group)
Use C-values for *k* = 0

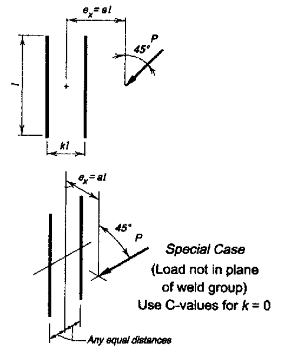
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18
0.100	4.04	4.05	4.05	4.05	4.06	4.07	4.07	4.08	4.08	4.09	4.09	4.09	4.09	4.08	4.08	4.07
0.150	3.83	3.83	3.83	3.83	3.84	3.84	3.84	3.85	3.86	3.87	3.88	3.89	3.91	3.92	3.93	3.93
0.200	3.63	3.63	3.64	3.65	3.65	3.66	3.67	3.68	3.68	3.70	3.71	3.73	3.74	3.76	3.78	3.79
0.250	3.43	3.43	3.43	3.44	3.46	3.48	3.49	3.51	3.52	3.54	3.56	3.58	3.60	3.62	3.64	3.66
0.300	3.21	3.21	3.22	3.24	3.27	3.29	3.32	3.35	3.37	3.39	3.41	3.45	3.48	3.50	3.52	3.54
0.400	2.81	2.81	2.83	2.86	2.90	2.94	2.99	3.03	3.07	3.10	3.14	3.19	3.24	3.28	3.31	3.34
0.500	2.46	2.46	2.48	2.52	2.58	2.63	2.69	2.75	2.80	2.85	2.89	2.96	3.02	3.08	3.12	3.16
0.600	2.17	2.17	2.20	2.24	2.30	2.37	2.44	2.50	2.56	2.61	2.66	2.75	2.83	2.89	2.95	2.99
0.700	1.93	1.93	1.96	2.01	2.08	2.15	2.22	2.29	2.35	2.41	2.47	2.57	2.65	2.72	2.78	2.84
0.800	1.73	1.73	1.77	1.82	1.89	1.96	2.03	2.10	2.17	2.24	2.30	2.40	2.49	2.57	2.64	2.70
0.900	1.56	1.57	1.60	1.66	1.73	1.80	1.87	1.95	2.02	2.08	2.14	2.25	2.34	2.43	2.50	2.57
1.00	1.43	1.43	1.47	1.52	1.59	1.66	1.74	1.81	1.88	1.95	2.01	2.12	2.22	2.30	2.38	2.45
1.20	1.21	1.22	1.25	1.31	1.37	1.44	1.51	1.59	1.65	1.72	1.78	1.89	1.99	2.08	2.16	2.23
1.40	1.05	1.06	1.09	1.14	1.20	1.27	1.34	1.41	1.47	1.53	1.59	1.70	1.81	1.90	1.98	2.05
1.60	0.924	0.932	0.964	1.01	1.07	1.13	1.20	1.26	1.32	1.39	1.44	1.55	1.65	1.74	1.82	1.90
1.80	0.825	0.833	0.864	0.908	0.961	1.02	1.08	1.14	1.20	1.26	1.32	1.42	1.52	1.61	1.69	1.76
2.00	0.745	0.753	0.783	0.823	0.873	0.928	0.987	1.04	1.10	1.15	1.21	1.31	1.41	1.49	1.57	1.65
2.20	0.680	0.688	0.715	0.752	0.799	0.851	0.905	0.960	1.01	1.07	1.12	1.22	1.31	1.39	1.47	1.54
2.40	0.625	0.632	0.657	0.693	0.736	0.785	0.837	0.889	0.940	0.991	1.04	1.13	1.22	1.30	1.38	1.45
2.60	0.577	0.585	0.608	0.641	0.683	0.728	0.777	0.827	0.876	0.924	0.971	1.06	1.15	1.23	1.30	1.37
2.80	0.537	0.544	0.565	0.597	0.636	0.679	0.725	0.772	0.819	0.865	0.911	0.997	1.08	1.16	1.23	1.30
3.00	0.501	0.509	0.529	0.559	0.595	0.636	0.680	0.724	0.769	0.813	0.856	0.940	1.02	1.09	1.16	1.23

Table 8–4 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



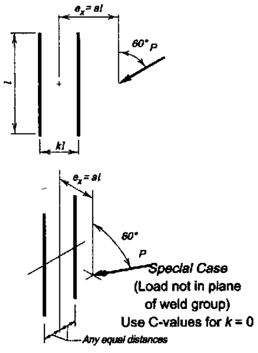
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64
0.100	4.49	4.49	4.50	4.51	4.53	4.55	4.57	4.59	4.61	4.62	4.64	4.66	4.68	4.69	4.69	4.70
0.150	4.18	4.18	4.20	4.23	4.26	4.30	4.34	4.37	4.40	4.43	4.46	4.51	4.55	4.57	4.60	4.62
0.200	3.92	3.92	3.93	3.96	3.99	4.03	4.08	4.13	4.18	4.22	4.26	4.33	4.39	4.44	4.47	4.51
0.250	3.69	3.70	3.71	3.74	3.77	3.81	3.86	3.91	3.95	4.01	4.06	4.14	4.22	4.28	4.33	4.38
0.300	3.48	3.48	3.50	3.53	3.57	3.62	3.67	3.72	3.77	3.81	3.86	3.96	4.05	4.12	4.18	4.24
0.400	3.09	3.09	3.12	3.16	3.21	3.27	3.33	3.39	3.45	3.50	3.55	3.64	3.73	3.82	3.90	3.97
0.500	2.75	2.75	2.78	2.83	2.89	2.96	3.03	3.10	3.17	3.23	3.29	3.39	3.48	3.57	3.65	3.72
0.600	2.46	2.46	2.49	2.55	2.62	2.69	2.77	2.85	2.92	3.00	3.06	3.17	3.27	3.36	3.43	3.51
0.700	2.21	2.22	2.25	2.31	2.38	2.46	2.55	2.63	2.71	2.78	2.85	2.98	3.08	3.17	3.26	3.33
0.800	2.00	2.01	2.05	2.11	2.18	2.27	2.35	2.44	2.52	2.60	2.67	2.80	2.91	3.01	3.09	3.17
0.900	1.83	1.83	1.87	1.93	2.01	2.09	2.18	2.27	2.35	2.43	2.50	2.64	2.75	2.85	2.95	3.03
1.00	1.68	1.69	1.72	1.79	1.86	1.95	2.03	2.12	2.20	2.28	2.36	2.49	2.61	2.72	2.81	2.89
1.20	1.43	1.44	1.48	1.55	1.62	1.70	1.79	1.87	1.95	2.03	2.11	2.24	2.36	2.47	2.57	2.66
1.40	1.25	1.26	1.30	1.36	1.43	1.51	1.59	1.67	1.75	1.83	1.90	2.03	2.15	2.26	2.36	2.45
1.60	1.11	1.12	1.15	1.21	1.28	1.35	1.43	1.51	1.58	1.66	1.72	1.86	1.98	2.09	2.19	2.28
1.80	0.993	1.00	1.04	1.09	1.15	1.22	1.30	1.37	1.44	1.51	1.58	1.71	1.83	1.93	2.03	2.12
2.00	0.900	0.909	0.943	0.991	1.05	1.11	1.18	1.25	1.32	1.39	1.46	1.58	1.69	1.80	1.90	1.99
2.20	0.823	0.832	0.863	0.908	0.963	1.02	1.09	1.16	1.22	1.29	1.35	1.47	1.58	1.68	1.78	1.87
2.40	0.757	0.765	0.795	0.837	0.889	0.947	1.01	1.07	1.13	1.20	1.25	1.37	1.48	1.58	1.67	1.76
2.60	0.701	0.709	0.737	0.777	0.825	0.880	0.937	0.999	1.06	1.12	1.17	1.28	1.39	1.49	1.58	1.66
2.80	0.652	0.661	0.687	0.725	0.771	0.823	0.877	0.935	0.991	1.05	1.10	1.21	1.31	1.40	1.49	1.58
3.00	0.611	0.619	0.643	0.679	0.723	0.772	0.824	0.877	0.932	0.985	1.04	1.14	1.24	1.33	1.42	1.50

Table 8–4 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



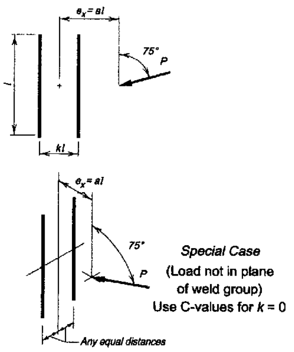
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11	5.11
0.100	4.87	4.87	4.90	4.94	4.99	5.03	5.07	5.10	5.12	5.13	5.14	5.15	5.15	5.15	5.15	5.15
0.150	4.61	4.62	4.65	4.71	4.77	4.84	4.91	4.97	5.01	5.05	5.07	5.11	5.12	5.13	5.14	5.14
0.200	4.36	4.37	4.41	4.46	4.54	4.63	4.71	4.79	4.87	4.92	4.97	5.03	5.07	5.09	5.11	5.12
0.250	4.13	4.13	4.17	4.23	4.31	4.40	4.51	4.61	4.70	4.78	4.84	4.94	5.00	5.04	5.07	5.09
0.300	3.93	3.94	3.97	4.02	4.10	4.19	4.30	4.41	4.52	4.62	4.70	4.83	4.91	4.97	5.02	5.04
0.400	3.57	3.58	3.62	3.67	3.75	3.83	3.93	4.03	4.15	4.27	4.39	4.57	4.71	4.81	4.88	4.93
0.500	3.26	3.27	3.31	3.37	3.45	3.54	3.64	3.74	3.83	3.95	4.07	4.29	4.47	4.61	4.72	4.79
0.600	2.98	2.99	3.03	3.10	3.19	3.28	3.38	3.49	3.58	3.69	3.78	4.02	4.22	4.39	4.53	4.63
0.700	2.73	2.74	2.79	2.86	2.95	3.05	3.16	3.26	3.37	3.46	3.57	3.76	3.97	4.16	4.32	4.45
0.800	2.52	2.53	2.57	2.65	2.74	2.85	2.95	3.06	3.17	3.27	3.37	3.55	3.74	3.94	4.11	4.26
0.900	2.33	2.34	2.39	2.47	2.56	2.67	2.77	2.88	2.99	3.09	3.19	3.37	3.54	3.73	3.91	4.07
1.00	2.17	2.18	2.23	2.30	2.40	2.50	2.61	2.72	2.82	2.93	3.03	3.21	3.37	3.54	3.72	3.88
1.20	1.89	1.90	1.95	2.03	2.12	2.22	2.33	2.44	2.54	2.65	2.74	2.93	3.09	3.24	3.39	3.54
1.40	1.67	1.68	1.73	1.81	1.89	1.99	2.10	2.20	2.31	2.41	2.50	2.68	2.85	2.99	3.13	3.27
1.60	1.49	1.51	1.55	1.63	1.71	1.80	1.91	2.01	2.11	2.20	2.30	2.47	2.63	2.78	2.92	3.05
1.80	1.35	1.36	1.41	1.47	1.56	1.65	1.74	1.84	1.93	2.03	2.12	2.29	2.45	2.60	2.73	2.86
2.00	1.23	1.24	1.29	1.35	1.42	1.51	1.60	1.69	1.79	1.88	1.97	2.13	2.29	2.43	2.56	2.69
2.20	1.13	1.14	1.18	1.24	1.31	1.39	1.48	1.57	1.66	1.75	1.83	1.99	2.14	2.28	2.41	2.54
2.40	1.04	1.05	1.09	1.15	1.22	1.29	1.38	1.46	1.55	1.63	1.71	1.87	2.01	2.15	2.28	2.40
2.60	0.968	0.979	1.02	1.07	1.14	1.21	1.29	1.37	1.45	1.53	1.61	1.76	1.90	2.03	2.16	2.28
2.80	0.903	0.915	0.949	1.00	1.06	1.13	1.21	1.28	1.36	1.44	1.51	1.66	1.80	1.93	2.05	2.17
3.00	0.847	0.857	0.891	0.939	0.999	1.07	1.14	1.21	1.28	1.36	1.43	1.57	1.70	1.83	1.95	2.06

Table 8–4 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi C_1 D l}$ $D_{min} = \frac{P_u}{\phi C C_1 l}$ $l_{min} = \frac{P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 D l}$ $D_{min} = \frac{\Omega P_a}{C C_1 l}$ $l_{min} = \frac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



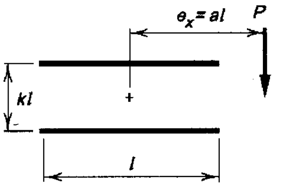
a	*k*															
	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44
0.100	5.17	5.20	5.26	5.33	5.39	5.42	5.44	5.45	5.46	5.46	5.46	5.46	5.46	5.45	5.45	5.45
0.150	5.01	5.03	5.10	5.20	5.28	5.35	5.39	5.42	5.43	5.44	5.45	5.45	5.45	5.45	5.45	5.45
0.200	4.85	4.87	4.95	5.06	5.17	5.26	5.32	5.37	5.39	5.42	5.43	5.44	5.45	5.45	5.45	5.45
0.250	4.71	4.72	4.80	4.92	5.04	5.15	5.24	5.30	5.35	5.38	5.40	5.42	5.44	5.44	5.45	5.45
0.300	4.57	4.59	4.65	4.78	4.92	5.04	5.15	5.23	5.29	5.33	5.36	5.40	5.42	5.43	5.44	5.44
0.400	4.32	4.33	4.39	4.51	4.67	4.82	4.96	5.07	5.15	5.22	5.27	5.34	5.38	5.40	5.42	5.43
0.500	4.09	4.11	4.17	4.27	4.43	4.60	4.76	4.89	5.00	5.09	5.16	5.26	5.32	5.36	5.38	5.40
0.600	3.88	3.90	3.96	4.06	4.21	4.38	4.56	4.71	4.85	4.95	5.04	5.17	5.25	5.31	5.34	5.37
0.700	3.69	3.70	3.76	3.87	4.01	4.18	4.36	4.53	4.68	4.81	4.91	5.07	5.17	5.24	5.29	5.33
0.800	3.50	3.52	3.59	3.69	3.83	3.99	4.17	4.35	4.51	4.66	4.78	4.96	5.08	5.17	5.24	5.28
0.900	3.34	3.35	3.42	3.53	3.66	3.81	3.99	4.18	4.35	4.50	4.64	4.85	4.99	5.10	5.18	5.23
1.00	3.18	3.20	3.26	3.37	3.50	3.65	3.83	4.01	4.19	4.35	4.50	4.73	4.90	5.02	5.11	5.18
1.20	2.90	2.91	2.98	3.09	3.22	3.36	3.52	3.70	3.88	4.06	4.22	4.49	4.70	4.85	4.97	5.06
1.40	2.65	2.67	2.74	2.84	2.97	3.11	3.27	3.43	3.61	3.78	3.95	4.25	4.49	4.67	4.81	4.93
1.60	2.44	2.46	2.53	2.63	2.75	2.89	3.04	3.19	3.35	3.53	3.70	4.01	4.27	4.48	4.65	4.78
1.80	2.25	2.27	2.34	2.44	2.56	2.69	2.83	2.98	3.14	3.30	3.47	3.79	4.06	4.29	4.48	4.63
2.00	2.09	2.11	2.18	2.27	2.39	2.51	2.65	2.80	2.95	3.10	3.26	3.58	3.86	4.10	4.31	4.48
2.20	1.95	1.97	2.03	2.12	2.23	2.36	2.49	2.63	2.77	2.92	3.07	3.38	3.66	3.92	4.14	4.32
2.40	1.82	1.84	1.90	1.99	2.10	2.22	2.35	2.48	2.62	2.76	2.90	3.20	3.48	3.74	3.97	4.17
2.60	1.71	1.73	1.79	1.87	1.98	2.09	2.22	2.35	2.48	2.62	2.75	3.04	3.31	3.57	3.80	4.01
2.80	1.61	1.63	1.69	1.77	1.87	1.98	2.10	2.23	2.35	2.49	2.61	2.88	3.16	3.41	3.65	3.86
3.00	1.52	1.54	1.59	1.67	1.77	1.88	1.99	2.12	2.24	2.36	2.49	2.75	3.01	3.26	3.49	3.71

Table 8–5
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ (ϕ = 0.75, Ω = 2.00)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



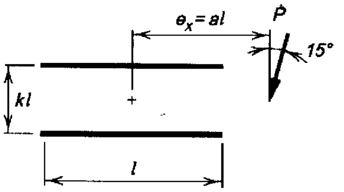
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57	5.57
0.100	4.31	4.36	4.48	4.65	4.82	4.98	5.11	5.21	5.29	5.35	5.40	5.45	5.49	5.51	5.52	5.53
0.150	3.90	3.94	4.04	4.20	4.39	4.58	4.75	4.90	5.02	5.12	5.20	5.31	5.38	5.43	5.46	5.48
0.200	3.53	3.57	3.66	3.80	3.99	4.20	4.40	4.57	4.73	4.86	4.97	5.14	5.24	5.32	5.37	5.41
0.250	3.21	3.25	3.34	3.47	3.64	3.84	4.06	4.26	4.43	4.59	4.72	4.93	5.08	5.19	5.27	5.32
0.300	2.93	2.97	3.06	3.18	3.34	3.52	3.74	3.95	4.14	4.32	4.47	4.72	4.91	5.05	5.15	5.22
0.400	2.48	2.51	2.59	2.71	2.85	3.01	3.19	3.40	3.61	3.81	3.99	4.29	4.54	4.73	4.87	4.99
0.500	2.13	2.16	2.24	2.34	2.47	2.62	2.78	2.95	3.14	3.35	3.54	3.88	4.16	4.39	4.58	4.73
0.600	1.86	1.89	1.96	2.05	2.17	2.31	2.45	2.61	2.77	2.95	3.15	3.50	3.81	4.06	4.28	4.46
0.700	1.65	1.67	1.74	1.82	1.93	2.05	2.19	2.33	2.48	2.64	2.81	3.17	3.48	3.75	3.99	4.19
0.800	1.48	1.50	1.56	1.64	1.74	1.85	1.97	2.10	2.24	2.38	2.53	2.87	3.19	3.46	3.71	3.93
0.900	1.34	1.36	1.41	1.49	1.58	1.68	1.79	1.91	2.04	2.17	2.31	2.61	2.92	3.20	3.45	3.68
1.00	1.22	1.24	1.29	1.36	1.44	1.54	1.64	1.75	1.87	1.99	2.12	2.39	2.69	2.97	3.22	3.45
1.20	1.04	1.05	1.09	1.15	1.23	1.31	1.40	1.50	1.60	1.71	1.82	2.05	2.29	2.56	2.81	3.03
1.40	0.899	0.913	0.951	1.00	1.07	1.14	1.22	1.31	1.40	1.49	1.59	1.79	2.00	2.24	2.47	2.69
1.60	0.792	0.805	0.839	0.887	0.945	1.01	1.08	1.16	1.24	1.32	1.41	1.59	1.78	1.98	2.19	2.40
1.80	0.708	0.721	0.751	0.793	0.847	0.907	0.972	1.04	1.11	1.19	1.27	1.43	1.59	1.77	1.96	2.16
2.00	0.641	0.652	0.679	0.717	0.765	0.821	0.881	0.944	1.01	1.08	1.15	1.29	1.45	1.61	1.77	1.95
2.20	0.585	0.595	0.620	0.656	0.699	0.749	0.805	0.863	0.924	0.987	1.05	1.19	1.33	1.47	1.62	1.78
2.40	0.539	0.547	0.569	0.603	0.644	0.689	0.740	0.795	0.851	0.908	0.969	1.09	1.22	1.35	1.49	1.64
2.60	0.499	0.505	0.528	0.559	0.596	0.639	0.685	0.736	0.789	0.843	0.899	1.01	1.13	1.25	1.38	1.51
2.80	0.464	0.471	0.491	0.520	0.555	0.595	0.639	0.685	0.735	0.785	0.837	0.945	1.06	1.17	1.29	1.41
3.00	0.433	0.440	0.459	0.485	0.519	0.556	0.597	0.641	0.688	0.736	0.784	0.885	0.989	1.10	1.21	1.32

Table 8–5 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



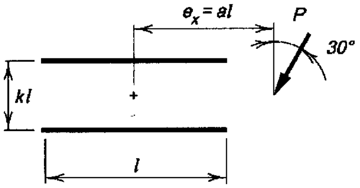
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44	5.44
0.100	4.38	4.39	4.45	4.58	4.73	4.88	5.01	5.11	5.19	5.25	5.30	5.36	5.39	5.41	5.42	5.43
0.150	3.96	3.98	4.04	4.14	4.29	4.47	4.64	4.79	4.91	5.01	5.09	5.21	5.28	5.33	5.36	5.38
0.200	3.60	3.62	3.68	3.79	3.92	4.09	4.27	4.45	4.60	4.74	4.85	5.02	5.13	5.21	5.27	5.31
0.250	3.28	3.30	3.37	3.47	3.61	3.76	3.94	4.12	4.29	4.45	4.59	4.81	4.96	5.07	5.15	5.22
0.300	3.00	3.02	3.09	3.20	3.33	3.47	3.64	3.82	4.00	4.17	4.33	4.59	4.78	4.92	5.03	5.11
0.400	2.55	2.57	2.64	2.74	2.87	3.01	3.15	3.31	3.49	3.66	3.83	4.13	4.39	4.58	4.74	4.86
0.500	2.20	2.22	2.28	2.38	2.50	2.63	2.77	2.92	3.07	3.23	3.40	3.71	3.99	4.23	4.42	4.58
0.600	1.92	1.94	2.00	2.10	2.20	2.33	2.46	2.60	2.74	2.89	3.04	3.35	3.63	3.88	4.10	4.29
0.700	1.70	1.72	1.78	1.87	1.97	2.08	2.21	2.34	2.47	2.61	2.74	3.03	3.30	3.56	3.79	4.00
0.800	1.53	1.55	1.60	1.68	1.77	1.89	2.00	2.12	2.25	2.37	2.50	2.76	3.02	3.27	3.51	3.72
0.900	1.38	1.40	1.45	1.53	1.61	1.72	1.83	1.94	2.06	2.18	2.29	2.53	2.78	3.02	3.24	3.46
1.00	1.26	1.28	1.32	1.39	1.48	1.57	1.68	1.78	1.89	2.01	2.12	2.34	2.56	2.79	3.01	3.22
1.20	1.07	1.09	1.13	1.19	1.26	1.35	1.44	1.53	1.63	1.73	1.83	2.03	2.23	2.43	2.63	2.82
1.40	0.929	0.943	0.980	1.03	1.10	1.17	1.26	1.34	1.43	1.52	1.61	1.79	1.97	2.14	2.32	2.50
1.60	0.820	0.832	0.865	0.915	0.975	1.04	1.11	1.19	1.27	1.35	1.43	1.60	1.76	1.92	2.08	2.24
1.80	0.733	0.745	0.775	0.819	0.873	0.935	1.00	1.07	1.14	1.22	1.29	1.44	1.59	1.74	1.88	2.03
2.00	0.663	0.673	0.701	0.741	0.791	0.847	0.908	0.972	1.04	1.11	1.17	1.31	1.45	1.59	1.72	1.85
2.20	0.605	0.615	0.640	0.677	0.723	0.773	0.831	0.889	0.951	1.01	1.08	1.21	1.33	1.46	1.59	1.71
2.40	0.557	0.565	0.589	0.623	0.665	0.712	0.764	0.820	0.876	0.933	0.993	1.11	1.23	1.35	1.47	1.58
2.60	0.516	0.523	0.545	0.577	0.616	0.660	0.708	0.760	0.812	0.867	0.921	1.03	1.15	1.26	1.37	1.47
2.80	0.480	0.487	0.508	0.537	0.573	0.615	0.659	0.707	0.757	0.808	0.859	0.965	1.07	1.17	1.28	1.38
3.00	0.448	0.455	0.475	0.503	0.536	0.575	0.617	0.661	0.709	0.757	0.805	0.904	1.00	1.10	1.20	1.30

Table 8–5 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10	5.10
0.100	4.49	4.50	4.54	4.59	4.66	4.74	4.82	4.89	4.95	4.99	5.03	5.07	5.10	5.12	5.13	5.13
0.150	4.09	4.10	4.13	4.18	4.27	4.36	4.46	4.57	4.67	4.75	4.82	4.92	4.99	5.03	5.06	5.08
0.200	3.76	3.77	3.80	3.86	3.93	4.01	4.11	4.23	4.35	4.46	4.56	4.72	4.83	4.90	4.96	5.00
0.250	3.47	3.47	3.51	3.57	3.64	3.73	3.83	3.92	4.04	4.16	4.28	4.48	4.64	4.75	4.83	4.89
0.300	3.20	3.21	3.25	3.31	3.40	3.49	3.59	3.68	3.78	3.89	4.01	4.24	4.43	4.57	4.68	4.76
0.400	2.75	2.76	2.81	2.88	2.97	3.07	3.17	3.28	3.38	3.48	3.58	3.77	3.99	4.18	4.34	4.47
0.500	2.39	2.41	2.45	2.52	2.62	2.72	2.83	2.94	3.05	3.15	3.25	3.43	3.60	3.80	3.98	4.13
0.600	2.11	2.12	2.17	2.24	2.34	2.44	2.55	2.66	2.77	2.87	2.97	3.15	3.31	3.47	3.65	3.81
0.700	1.87	1.89	1.94	2.01	2.11	2.21	2.31	2.42	2.53	2.63	2.73	2.91	3.07	3.22	3.37	3.52
0.800	1.68	1.70	1.75	1.82	1.91	2.01	2.11	2.22	2.32	2.42	2.52	2.70	2.86	3.01	3.15	3.29
0.900	1.53	1.54	1.59	1.66	1.75	1.84	1.94	2.04	2.15	2.24	2.34	2.51	2.68	2.82	2.96	3.09
1.00	1.40	1.41	1.45	1.52	1.61	1.70	1.79	1.89	1.99	2.09	2.18	2.35	2.51	2.66	2.79	2.92
1.20	1.19	1.20	1.24	1.31	1.38	1.46	1.55	1.64	1.74	1.83	1.91	2.07	2.23	2.37	2.50	2.63
1.40	1.03	1.04	1.08	1.14	1.21	1.28	1.37	1.45	1.53	1.62	1.70	1.85	2.00	2.14	2.26	2.38
1.60	0.912	0.923	0.959	1.01	1.07	1.14	1.22	1.30	1.37	1.45	1.53	1.67	1.81	1.94	2.07	2.18
1.80	0.816	0.827	0.859	0.905	0.964	1.03	1.10	1.17	1.24	1.31	1.38	1.52	1.65	1.78	1.90	2.01
2.00	0.739	0.749	0.779	0.821	0.873	0.933	0.999	1.07	1.13	1.20	1.27	1.39	1.52	1.64	1.75	1.86
2.20	0.675	0.684	0.712	0.751	0.800	0.855	0.915	0.977	1.04	1.10	1.17	1.29	1.41	1.52	1.63	1.73
2.40	0.620	0.629	0.655	0.692	0.737	0.788	0.844	0.901	0.960	1.02	1.08	1.19	1.31	1.41	1.52	1.62
2.60	0.575	0.583	0.607	0.641	0.683	0.731	0.783	0.837	0.892	0.948	1.00	1.11	1.22	1.32	1.42	1.52
2.80	0.535	0.541	0.564	0.597	0.636	0.681	0.729	0.780	0.833	0.885	0.939	1.04	1.14	1.24	1.34	1.43
3.00	0.500	0.507	0.528	0.559	0.596	0.637	0.684	0.731	0.781	0.831	0.881	0.979	1.08	1.17	1.26	1.35

Table 8–5 (continued)

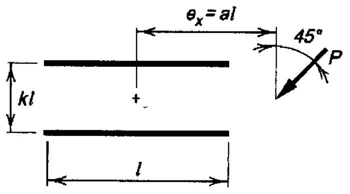
Coefficients *C* for Eccentrically Loaded Weld Groups

Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x/l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



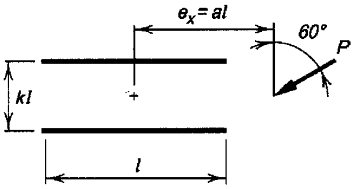
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64	4.64
0.100	4.49	4.49	4.50	4.51	4.53	4.55	4.57	4.59	4.61	4.62	4.64	4.66	4.67	4.69	4.69	4.70
0.150	4.18	4.18	4.20	4.23	4.26	4.30	4.34	4.37	4.40	4.43	4.46	4.51	4.55	4.58	4.60	4.62
0.200	3.92	3.92	3.93	3.96	3.99	4.03	4.08	4.13	4.18	4.22	4.26	4.33	4.39	4.44	4.47	4.51
0.250	3.69	3.70	3.71	3.74	3.77	3.81	3.86	3.91	3.95	4.01	4.06	4.14	4.22	4.28	4.33	4.38
0.300	3.48	3.48	3.50	3.53	3.57	3.62	3.67	3.72	3.77	3.81	3.86	3.96	4.05	4.12	4.18	4.24
0.400	3.09	3.09	3.12	3.16	3.21	3.27	3.33	3.39	3.45	3.50	3.55	3.64	3.73	3.82	3.90	3.97
0.500	2.75	2.75	2.78	2.83	2.89	2.96	3.03	3.10	3.17	3.23	3.29	3.39	3.48	3.57	3.65	3.72
0.600	2.46	2.46	2.49	2.55	2.62	2.69	2.77	2.85	2.92	3.00	3.06	3.17	3.27	3.36	3.43	3.51
0.700	2.21	2.22	2.25	2.31	2.38	2.46	2.55	2.63	2.71	2.78	2.85	2.98	3.08	3.17	3.26	3.33
0.800	2.00	2.01	2.05	2.11	2.18	2.27	2.35	2.44	2.52	2.60	2.67	2.80	2.91	3.01	3.09	3.17
0.900	1.83	1.83	1.87	1.93	2.01	2.09	2.18	2.27	2.35	2.43	2.50	2.64	2.75	2.85	2.95	3.03
1.00	1.68	1.69	1.72	1.79	1.86	1.95	2.03	2.12	2.20	2.28	2.36	2.49	2.61	2.72	2.81	2.89
1.20	1.43	1.44	1.48	1.55	1.62	1.70	1.79	1.87	1.95	2.03	2.11	2.24	2.36	2.47	2.57	2.66
1.40	1.25	1.26	1.30	1.36	1.43	1.51	1.59	1.67	1.75	1.83	1.90	2.03	2.15	2.26	2.36	2.45
1.60	1.11	1.12	1.15	1.21	1.28	1.35	1.43	1.51	1.58	1.66	1.72	1.86	1.98	2.09	2.19	2.28
1.80	0.993	1.00	1.04	1.09	1.15	1.22	1.30	1.37	1.44	1.51	1.58	1.71	1.83	1.93	2.03	2.12
2.00	0.900	0.909	0.943	0.991	1.05	1.11	1.18	1.25	1.32	1.39	1.46	1.58	1.69	1.80	1.90	1.99
2.20	0.823	0.832	0.863	0.908	0.963	1.02	1.09	1.16	1.22	1.29	1.35	1.47	1.58	1.68	1.78	1.87
2.40	0.757	0.765	0.795	0.837	0.889	0.947	1.01	1.07	1.13	1.20	1.25	1.37	1.48	1.58	1.67	1.76
2.60	0.701	0.709	0.737	0.777	0.825	0.880	0.937	0.999	1.06	1.12	1.17	1.28	1.39	1.49	1.58	1.66
2.80	0.652	0.661	0.687	0.725	0.771	0.823	0.877	0.935	0.991	1.05	1.10	1.21	1.31	1.40	1.49	1.58
3.00	0.611	0.619	0.643	0.679	0.723	0.772	0.824	0.877	0.932	0.985	1.04	1.14	1.24	1.33	1.42	1.50

Table 8–5 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



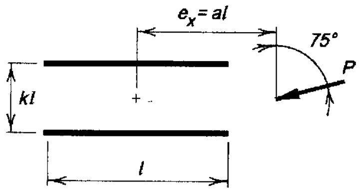
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18	4.18
0.100	4.26	4.26	4.26	4.26	4.26	4.25	4.25	4.25	4.24	4.24	4.23	4.22	4.21	4.20	4.19	4.18
0.150	4.13	4.13	4.13	4.13	4.13	4.13	4.14	4.14	4.14	4.14	4.14	4.13	4.13	4.12	4.11	4.10
0.200	3.97	3.97	3.97	3.97	3.98	3.98	3.99	4.00	4.01	4.02	4.02	4.03	4.03	4.03	4.03	4.03
0.250	3.85	3.85	3.85	3.85	3.86	3.86	3.87	3.87	3.88	3.89	3.90	3.92	3.93	3.94	3.94	3.95
0.300	3.74	3.74	3.74	3.75	3.75	3.76	3.76	3.77	3.78	3.78	3.79	3.81	3.83	3.85	3.86	3.86
0.400	3.50	3.51	3.51	3.52	3.53	3.55	3.56	3.57	3.59	3.60	3.61	3.63	3.65	3.67	3.69	3.71
0.500	3.26	3.26	3.27	3.29	3.31	3.33	3.36	3.38	3.41	3.42	3.44	3.48	3.50	3.53	3.55	3.57
0.600	3.02	3.02	3.03	3.05	3.09	3.13	3.16	3.20	3.22	3.25	3.28	3.33	3.37	3.40	3.42	3.45
0.700	2.79	2.79	2.81	2.84	2.88	2.93	2.97	3.02	3.06	3.09	3.12	3.18	3.23	3.27	3.31	3.33
0.800	2.58	2.58	2.61	2.64	2.69	2.75	2.80	2.85	2.90	2.94	2.98	3.05	3.11	3.15	3.19	3.23
0.900	2.39	2.40	2.42	2.46	2.52	2.58	2.64	2.69	2.75	2.80	2.84	2.92	2.98	3.04	3.09	3.13
1.00	2.23	2.23	2.26	2.30	2.36	2.43	2.49	2.55	2.61	2.66	2.71	2.80	2.87	2.93	2.99	3.03
1.20	1.94	1.95	1.98	2.03	2.09	2.16	2.23	2.30	2.37	2.43	2.48	2.58	2.66	2.73	2.79	2.85
1.40	1.71	1.72	1.75	1.81	1.87	1.94	2.02	2.09	2.16	2.23	2.28	2.39	2.48	2.56	2.63	2.68
1.60	1.53	1.54	1.57	1.62	1.69	1.77	1.84	1.91	1.98	2.05	2.11	2.22	2.31	2.40	2.47	2.54
1.80	1.38	1.39	1.42	1.47	1.54	1.61	1.69	1.76	1.83	1.90	1.96	2.07	2.17	2.25	2.33	2.40
2.00	1.25	1.26	1.29	1.35	1.41	1.49	1.56	1.63	1.70	1.76	1.82	1.94	2.04	2.13	2.21	2.28
2.20	1.15	1.16	1.19	1.24	1.31	1.37	1.45	1.52	1.59	1.65	1.71	1.82	1.92	2.01	2.09	2.17
2.40	1.06	1.07	1.10	1.15	1.21	1.28	1.35	1.42	1.48	1.55	1.61	1.72	1.82	1.91	1.99	2.07
2.60	0.981	0.991	1.02	1.07	1.13	1.19	1.26	1.33	1.39	1.46	1.51	1.62	1.72	1.81	1.90	1.97
2.80	0.915	0.924	0.955	1.00	1.06	1.12	1.19	1.25	1.31	1.37	1.43	1.54	1.64	1.73	1.81	1.89
3.00	0.857	0.865	0.896	0.941	0.995	1.06	1.12	1.18	1.24	1.30	1.36	1.46	1.56	1.65	1.73	1.81

Table 8–5 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84	3.84
0.100	3.81	3.81	3.82	3.83	3.83	3.84	3.84	3.85	3.85	3.85	3.85	3.84	3.82	3.81	3.79	3.40
0.150	3.85	3.85	3.86	3.86	3.86	3.86	3.85	3.85	3.84	3.84	3.83	3.81	3.80	3.78	3.76	3.74
0.200	3.84	3.84	3.84	3.83	3.83	3.83	3.82	3.82	3.81	3.80	3.80	3.78	3.76	3.75	3.73	3.71
0.250	3.83	3.83	3.83	3.82	3.82	3.81	3.80	3.80	3.79	3.78	3.77	3.75	3.74	3.72	3.70	3.68
0.300	3.81	3.81	3.81	3.80	3.80	3.79	3.78	3.77	3.77	3.76	3.75	3.73	3.71	3.69	3.67	3.66
0.400	3.78	3.78	3.77	3.76	3.75	3.74	3.73	3.72	3.71	3.70	3.69	3.67	3.66	3.64	3.62	3.61
0.500	3.72	3.72	3.71	3.70	3.69	3.68	3.67	3.66	3.65	3.64	3.64	3.62	3.60	3.59	3.57	3.56
0.600	3.65	3.64	3.64	3.63	3.62	3.61	3.60	3.59	3.58	3.58	3.57	3.56	3.55	3.53	3.52	3.51
0.700	3.55	3.55	3.54	3.54	3.54	3.53	3.52	3.52	3.51	3.51	3.50	3.49	3.49	3.48	3.47	3.46
0.800	3.45	3.45	3.45	3.44	3.44	3.44	3.44	3.44	3.44	3.43	3.43	3.43	3.42	3.42	3.41	3.41
0.900	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.35	3.36	3.36	3.36	3.36	3.36	3.36	3.36
1.00	3.23	3.23	3.23	3.24	3.25	3.25	3.26	3.26	3.27	3.28	3.28	3.29	3.30	3.30	3.31	3.30
1.20	3.00	3.00	3.01	3.02	3.04	3.06	3.08	3.09	3.11	3.12	3.13	3.16	3.17	3.19	3.20	3.21
1.40	2.77	2.77	2.78	2.81	2.84	2.87	2.90	2.93	2.95	2.97	2.99	3.02	3.05	3.07	3.09	3.11
1.60	2.57	2.57	2.58	2.61	2.65	2.69	2.73	2.77	2.80	2.83	2.85	2.90	2.93	2.96	2.99	3.01
1.80	2.38	2.38	2.40	2.43	2.48	2.52	2.57	2.62	2.65	2.69	2.72	2.78	2.82	2.86	2.89	2.92
2.00	2.21	2.21	2.23	2.27	2.32	2.37	2.43	2.48	2.52	2.56	2.60	2.66	2.72	2.76	2.80	2.83
2.20	2.05	2.06	2.08	2.13	2.18	2.24	2.30	2.35	2.40	2.44	2.49	2.56	2.62	2.67	2.71	2.74
2.40	1.92	1.92	1.95	1.99	2.05	2.11	2.18	2.23	2.29	2.33	2.38	2.46	2.52	2.58	2.62	2.66
2.60	1.79	1.80	1.83	1.88	1.94	2.00	2.07	2.13	2.18	2.23	2.28	2.36	2.43	2.49	2.54	2.59
2.80	1.69	1.69	1.72	1.77	1.83	1.90	1.96	2.03	2.09	2.14	2.19	2.27	2.35	2.41	2.47	2.51
3.00	1.59	1.59	1.63	1.68	1.74	1.81	1.87	1.94	2.00	2.05	2.10	2.19	2.27	2.33	2.39	2.44

Table 8–6
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

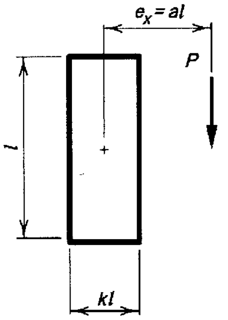
$R_n = CC_1Dl$ ($\phi = 0.75$ $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where

P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



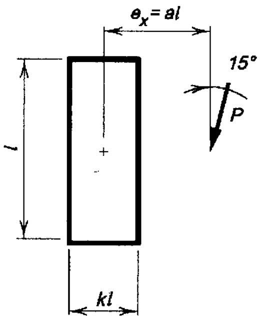
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.06	3.62	4.18	4.73	5.29	5.85	6.40	6.96	7.52	8.07	8.63	9.74	10.9	12.0	13.1	14.2
0.100	3.72	4.10	4.56	5.03	5.53	6.04	6.55	7.06	7.58	8.10	8.62	9.66	10.7	11.8	12.8	13.9
0.150	3.67	4.06	4.49	4.94	5.41	5.89	6.37	6.86	7.36	7.86	8.35	9.36	10.4	11.4	12.4	13.4
0.200	3.51	3.93	4.34	4.76	5.21	5.66	6.12	6.59	7.07	7.54	8.03	9.00	10.0	11.0	12.0	13.0
0.250	3.31	3.72	4.13	4.54	4.96	5.39	5.83	6.28	6.74	7.20	7.67	8.61	9.57	10.5	11.5	12.5
0.300	3.09	3.48	3.89	4.29	4.69	5.11	5.53	5.97	6.41	6.85	7.31	8.23	9.17	10.1	11.1	12.1
0.400	2.66	3.01	3.38	3.77	4.16	4.54	4.94	5.35	5.76	6.18	6.61	7.50	8.40	9.33	10.3	11.2
0.500	2.29	2.60	2.94	3.30	3.67	4.04	4.41	4.79	5.18	5.59	6.00	6.84	7.71	8.61	9.52	10.5
0.600	2.00	2.26	2.57	2.90	3.25	3.60	3.95	4.31	4.69	5.07	5.46	6.27	7.11	7.97	8.86	9.76
0.700	1.76	1.99	2.27	2.57	2.90	3.24	3.57	3.91	4.26	4.62	4.99	5.77	6.57	7.41	8.27	9.15
0.800	1.56	1.78	2.02	2.30	2.60	2.93	3.24	3.57	3.90	4.24	4.60	5.33	6.10	6.90	7.73	8.58
0.900	1.41	1.60	1.82	2.08	2.36	2.67	2.97	3.27	3.58	3.91	4.25	4.95	5.68	6.45	7.25	8.07
1.00	1.28	1.45	1.66	1.89	2.16	2.45	2.73	3.02	3.31	3.62	3.94	4.60	5.30	6.04	6.80	7.59
1.20	1.07	1.22	1.40	1.61	1.84	2.09	2.35	2.61	2.87	3.15	3.43	4.03	4.67	5.33	6.04	6.77
1.40	0.927	1.05	1.21	1.40	1.60	1.83	2.06	2.29	2.53	2.77	3.03	3.58	4.15	4.77	5.41	6.09
1.60	0.815	0.925	1.07	1.23	1.42	1.62	1.83	2.04	2.25	2.48	2.71	3.21	3.74	4.30	4.90	5.53
1.80	0.725	0.825	0.952	1.10	1.27	1.45	1.64	1.83	2.03	2.23	2.45	2.90	3.39	3.91	4.47	5.05
2.00	0.655	0.744	0.860	0.995	1.15	1.31	1.49	1.66	1.85	2.03	2.23	2.65	3.10	3.59	4.10	4.65
2.20	0.596	0.679	0.784	0.907	1.05	1.20	1.36	1.52	1.69	1.87	2.05	2.44	2.86	3.31	3.79	4.30
2.40	0.547	0.623	0.720	0.833	0.961	1.10	1.25	1.41	1.56	1.72	1.89	2.25	2.65	3.07	3.52	4.00
2.60	0.505	0.575	0.665	0.771	0.889	1.02	1.16	1.30	1.45	1.60	1.76	2.10	2.47	2.86	3.29	3.73
2.80	0.469	0.535	0.619	0.717	0.828	0.949	1.08	1.21	1.35	1.49	1.64	1.96	2.31	2.68	3.08	3.50
3.00	0.439	0.500	0.579	0.671	0.773	0.888	1.01	1.14	1.27	1.40	1.54	1.84	2.16	2.52	2.89	3.29

Table 8–6 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x/l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



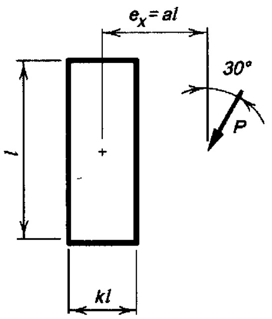
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.85	4.39	4.94	5.48	6.03	6.57	7.12	7.66	8.21	8.75	9.30	10.4	11.5	12.6	13.7	14.7
0.100	3.79	4.22	4.69	5.19	5.70	6.21	6.73	7.24	7.77	8.30	8.82	9.88	10.9	12.0	13.0	14.1
0.150	3.68	4.13	4.59	5.05	5.53	6.01	6.49	6.98	7.48	7.97	8.47	9.47	10.5	11.5	12.5	13.6
0.200	3.51	3.95	4.40	4.85	5.31	5.76	6.23	6.69	7.16	7.64	8.12	9.09	10.1	11.1	12.1	13.1
0.250	3.31	3.72	4.15	4.60	5.04	5.48	5.93	6.38	6.83	7.30	7.76	8.70	9.66	10.6	11.6	12.6
0.300	3.09	3.48	3.89	4.33	4.76	5.19	5.62	6.06	6.50	6.95	7.40	8.32	9.26	10.2	11.2	12.2
0.400	2.67	3.01	3.39	3.79	4.20	4.61	5.02	5.44	5.86	6.28	6.71	7.60	8.51	9.43	10.4	11.3
0.500	2.32	2.62	2.95	3.31	3.70	4.10	4.49	4.88	5.28	5.69	6.10	6.95	7.83	8.72	9.64	10.6
0.600	2.02	2.29	2.59	2.92	3.27	3.65	4.03	4.40	4.78	5.17	5.57	6.38	7.23	8.10	8.99	9.90
0.700	1.79	2.02	2.29	2.59	2.92	3.28	3.64	4.00	4.36	4.73	5.11	5.89	6.70	7.54	8.41	9.29
0.800	1.59	1.81	2.05	2.33	2.64	2.97	3.31	3.65	3.99	4.35	4.70	5.45	6.23	7.04	7.87	8.73
0.900	1.44	1.63	1.85	2.11	2.39	2.71	3.03	3.35	3.68	4.01	4.35	5.06	5.81	6.58	7.39	8.21
1.00	1.31	1.48	1.69	1.93	2.20	2.49	2.79	3.10	3.40	3.72	4.04	4.72	5.43	6.17	6.94	7.74
1.20	1.10	1.25	1.43	1.64	1.88	2.13	2.41	2.68	2.95	3.24	3.53	4.14	4.78	5.47	6.18	6.92
1.40	0.952	1.08	1.24	1.43	1.64	1.87	2.11	2.35	2.60	2.85	3.12	3.68	4.27	4.89	5.55	6.24
1.60	0.837	0.951	1.09	1.26	1.45	1.65	1.87	2.10	2.32	2.55	2.79	3.30	3.85	4.42	5.03	5.67
1.80	0.747	0.848	0.977	1.13	1.30	1.48	1.68	1.89	2.09	2.30	2.52	2.99	3.49	4.03	4.59	5.19
2.00	0.675	0.767	0.884	1.02	1.18	1.35	1.53	1.72	1.90	2.10	2.30	2.73	3.20	3.70	4.22	4.78
2.20	0.615	0.699	0.807	0.932	1.07	1.23	1.40	1.57	1.75	1.93	2.11	2.52	2.95	3.41	3.90	4.42
2.40	0.564	0.641	0.741	0.857	0.988	1.13	1.29	1.45	1.61	1.78	1.95	2.33	2.73	3.17	3.63	4.12
2.60	0.521	0.593	0.685	0.793	0.915	1.05	1.19	1.34	1.49	1.65	1.82	2.17	2.55	2.95	3.39	3.85
2.80	0.484	0.552	0.637	0.739	0.851	0.976	1.11	1.25	1.39	1.54	1.70	2.03	2.38	2.77	3.17	3.61
3.00	0.452	0.515	0.596	0.691	0.796	0.912	1.04	1.17	1.31	1.45	1.59	1.90	2.24	2.60	2.99	3.40

Table 8–6 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ (ϕ = 0.75, Ω = 2.00)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



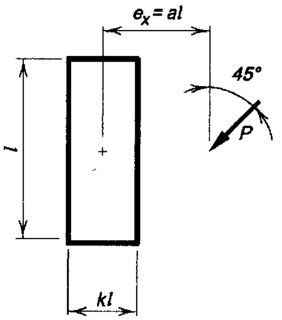
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.37	4.89	5.40	5.91	6.43	6.94	7.46	7.97	8.49	9.00	9.52	10.5	11.6	12.6	13.6	14.7
0.100	4.04	4.60	5.13	5.65	6.16	6.67	7.17	7.68	8.19	8.69	9.20	10.2	11.3	12.3	13.3	14.4
0.150	3.83	4.33	4.85	5.36	5.86	6.36	6.86	7.35	7.85	8.35	8.85	9.85	10.9	11.9	12.9	14.0
0.200	3.63	4.09	4.57	5.06	5.55	6.04	6.52	6.99	7.48	7.97	8.46	9.45	10.5	11.5	12.5	13.6
0.250	3.43	3.85	4.29	4.76	5.24	5.72	6.19	6.66	7.12	7.59	8.06	9.03	10.0	11.0	12.1	13.1
0.300	3.21	3.60	4.02	4.47	4.93	5.39	5.87	6.33	6.78	7.24	7.70	8.64	9.61	10.6	11.6	12.6
0.400	2.81	3.15	3.52	3.93	4.35	4.79	5.25	5.71	6.15	6.59	7.03	7.93	8.86	9.81	10.8	11.8
0.500	2.46	2.76	3.10	3.47	3.86	4.27	4.71	5.14	5.58	6.01	6.43	7.31	8.21	9.13	10.1	11.0
0.600	2.17	2.44	2.74	3.08	3.45	3.84	4.24	4.66	5.09	5.50	5.91	6.76	7.63	8.53	9.45	10.4
0.700	1.93	2.17	2.45	2.76	3.10	3.47	3.85	4.25	4.67	5.06	5.45	6.27	7.11	7.98	8.87	9.79
0.800	1.73	1.95	2.21	2.49	2.82	3.16	3.53	3.91	4.29	4.67	5.05	5.83	6.64	7.48	8.35	9.24
0.900	1.56	1.77	2.00	2.27	2.57	2.90	3.24	3.61	3.97	4.33	4.69	5.44	6.22	7.03	7.87	8.73
1.00	1.43	1.61	1.83	2.08	2.37	2.68	3.00	3.34	3.69	4.03	4.37	5.09	5.84	6.62	7.43	8.27
1.20	1.21	1.37	1.56	1.79	2.04	2.31	2.60	2.91	3.22	3.53	3.84	4.50	5.19	5.91	6.67	7.45
1.40	1.05	1.19	1.36	1.56	1.78	2.03	2.29	2.57	2.85	3.13	3.41	4.01	4.65	5.32	6.03	6.76
1.60	0.924	1.05	1.20	1.38	1.58	1.81	2.04	2.29	2.55	2.80	3.07	3.62	4.21	4.83	5.49	6.17
1.80	0.825	0.937	1.08	1.24	1.42	1.63	1.84	2.07	2.30	2.54	2.78	3.29	3.84	4.42	5.03	5.67
2.00	0.745	0.847	0.975	1.13	1.29	1.48	1.67	1.88	2.10	2.31	2.54	3.01	3.52	4.06	4.63	5.24
2.20	0.680	0.772	0.891	1.03	1.18	1.35	1.53	1.73	1.93	2.13	2.34	2.78	3.25	3.76	4.29	4.86
2.40	0.625	0.711	0.819	0.947	1.09	1.25	1.41	1.59	1.78	1.97	2.16	2.58	3.02	3.49	4.00	4.53
2.60	0.577	0.657	0.759	0.877	1.01	1.16	1.31	1.48	1.65	1.83	2.01	2.40	2.82	3.26	3.74	4.24
2.80	0.537	0.611	0.705	0.817	0.941	1.08	1.22	1.38	1.54	1.71	1.88	2.24	2.64	3.06	3.51	3.98
3.00	0.501	0.571	0.660	0.764	0.881	1.01	1.15	1.29	1.45	1.60	1.76	2.11	2.48	2.88	3.30	3.76

Table 8–6 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



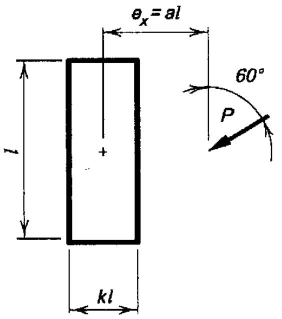
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.68	5.15	5.62	6.09	6.55	7.02	7.49	7.96	8.43	8.89	9.36	10.3	11.2	12.2	13.1	14.0
0.100	4.49	4.99	5.48	5.97	6.45	6.94	7.43	7.93	8.42	8.90	9.39	10.4	11.4	12.3	13.3	14.3
0.150	4.18	4.69	5.19	5.67	6.16	6.65	7.15	7.65	8.15	8.65	9.15	10.1	11.1	12.1	13.1	14.1
0.200	3.92	4.39	4.87	5.36	5.84	6.33	6.83	7.33	7.84	8.34	8.85	9.86	10.9	11.9	12.9	13.9
0.250	3.69	4.13	4.58	5.05	5.52	6.01	6.50	7.00	7.51	8.02	8.53	9.54	10.6	11.6	12.6	13.6
0.300	3.48	3.88	4.31	4.76	5.22	5.69	6.18	6.67	7.17	7.68	8.20	9.21	10.2	11.3	12.3	13.3
0.400	3.09	3.45	3.83	4.25	4.68	5.13	5.59	6.07	6.56	7.06	7.56	8.55	9.57	10.6	11.6	12.7
0.500	2.75	3.07	3.42	3.81	4.22	4.65	5.09	5.55	6.03	6.51	7.01	7.95	8.94	10.0	11.0	12.0
0.600	2.46	2.75	3.07	3.43	3.82	4.23	4.66	5.11	5.57	6.04	6.52	7.43	8.38	9.36	10.4	11.4
0.700	2.21	2.48	2.78	3.11	3.48	3.88	4.29	4.72	5.17	5.62	6.08	6.96	7.87	8.82	9.80	10.8
0.800	2.00	2.25	2.53	2.85	3.19	3.57	3.97	4.38	4.81	5.24	5.69	6.54	7.42	8.33	9.29	10.3
0.900	1.83	2.05	2.31	2.61	2.95	3.31	3.68	4.08	4.49	4.91	5.33	6.16	7.00	7.89	8.81	9.76
1.00	1.68	1.88	2.13	2.41	2.73	3.07	3.43	3.81	4.20	4.60	5.01	5.81	6.63	7.48	8.37	9.30
1.20	1.43	1.62	1.84	2.09	2.38	2.69	3.01	3.35	3.71	4.08	4.46	5.20	5.97	6.77	7.59	8.46
1.40	1.25	1.41	1.61	1.84	2.10	2.38	2.68	2.99	3.31	3.65	4.00	4.69	5.41	6.16	6.94	7.75
1.60	1.11	1.25	1.43	1.64	1.88	2.13	2.40	2.69	2.99	3.30	3.62	4.26	4.94	5.64	6.38	7.14
1.80	0.993	1.12	1.29	1.48	1.69	1.93	2.18	2.44	2.72	3.00	3.30	3.90	4.53	5.20	5.89	6.61
2.00	0.900	1.02	1.17	1.35	1.54	1.76	1.99	2.23	2.49	2.75	3.03	3.59	4.18	4.80	5.46	6.14
2.20	0.823	0.932	1.07	1.23	1.42	1.61	1.83	2.05	2.29	2.54	2.79	3.32	3.88	4.47	5.09	5.73
2.40	0.757	0.859	0.988	1.14	1.31	1.49	1.69	1.90	2.12	2.35	2.59	3.09	3.61	4.17	4.75	5.37
2.60	0.701	0.796	0.916	1.06	1.21	1.39	1.57	1.77	1.98	2.19	2.42	2.89	3.38	3.91	4.46	5.04
2.80	0.652	0.741	0.855	0.985	1.13	1.30	1.47	1.65	1.85	2.05	2.27	2.70	3.17	3.67	4.20	4.75
3.00	0.611	0.693	0.800	0.924	1.06	1.21	1.38	1.55	1.74	1.93	2.13	2.54	2.99	3.46	3.97	4.50

Table 8–6 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ (ϕ = 0.75, Ω = 2.00)
or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi C_1 Dl}$ $D_{min} = \frac{P_u}{\phi CC_1 l}$ $l_{min} = \frac{P_u}{\phi CC_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 Dl}$ $D_{min} = \frac{\Omega P_a}{CC_1 l}$ $l_{min} = \frac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.15	5.58	6.02	6.46	6.89	7.33	7.77	8.20	8.64	9.08	9.52	10.4	11.3	12.1	13.0	13.9
0.100	4.87	5.29	5.74	6.19	6.66	7.13	7.60	8.07	8.53	8.98	9.44	10.3	11.2	12.1	13.0	13.9
0.150	4.61	5.04	5.48	5.93	6.40	6.89	7.37	7.86	8.34	8.82	9.29	10.2	11.1	12.0	12.9	13.9
0.200	4.36	4.79	5.23	5.67	6.14	6.63	7.13	7.62	8.12	8.61	9.10	10.1	11.0	11.9	12.8	13.7
0.250	4.13	4.55	4.99	5.43	5.88	6.37	6.87	7.38	7.89	8.39	8.89	9.87	10.8	11.8	12.7	13.6
0.300	3.93	4.33	4.76	5.19	5.64	6.12	6.62	7.13	7.65	8.16	8.67	9.67	10.6	11.6	12.5	13.5
0.400	3.57	3.94	4.34	4.76	5.20	5.66	6.15	6.66	7.17	7.69	8.21	9.24	10.2	11.2	12.2	13.2
0.500	3.26	3.60	3.98	4.38	4.81	5.27	5.74	6.24	6.75	7.26	7.78	8.81	9.82	10.8	11.8	12.8
0.600	2.98	3.30	3.65	4.05	4.47	4.92	5.38	5.86	6.36	6.87	7.38	8.41	9.43	10.4	11.4	12.4
0.700	2.73	3.03	3.37	3.75	4.16	4.60	5.05	5.52	6.00	6.50	7.00	8.03	9.05	10.1	11.1	12.1
0.800	2.52	2.80	3.12	3.49	3.89	4.31	4.75	5.21	5.68	6.16	6.65	7.66	8.68	9.70	10.7	11.7
0.900	2.33	2.60	2.90	3.25	3.64	4.05	4.47	4.92	5.38	5.85	6.33	7.31	8.32	9.32	10.3	11.3
1.00	2.17	2.42	2.71	3.05	3.42	3.81	4.23	4.66	5.10	5.56	6.02	6.99	7.98	8.96	9.94	10.9
1.20	1.89	2.11	2.38	2.69	3.03	3.40	3.79	4.19	4.61	5.04	5.48	6.40	7.33	8.27	9.24	10.2
1.40	1.67	1.87	2.12	2.41	2.72	3.06	3.42	3.80	4.19	4.60	5.02	5.89	6.76	7.66	8.59	9.54
1.60	1.49	1.68	1.91	2.17	2.46	2.78	3.11	3.47	3.83	4.22	4.61	5.43	6.26	7.11	8.01	8.93
1.80	1.35	1.52	1.73	1.98	2.25	2.54	2.85	3.18	3.53	3.89	4.26	5.04	5.82	6.63	7.48	8.37
2.00	1.23	1.39	1.58	1.81	2.06	2.34	2.63	2.94	3.26	3.60	3.95	4.69	5.43	6.20	7.01	7.86
2.20	1.13	1.27	1.46	1.67	1.91	2.16	2.43	2.73	3.03	3.35	3.68	4.38	5.09	5.82	6.59	7.40
2.40	1.04	1.18	1.35	1.55	1.77	2.01	2.27	2.54	2.83	3.13	3.44	4.10	4.79	5.48	6.21	6.99
2.60	0.968	1.09	1.26	1.44	1.65	1.88	2.12	2.38	2.65	2.93	3.23	3.85	4.51	5.17	5.87	6.61
2.80	0.903	1.02	1.17	1.35	1.55	1.76	1.99	2.23	2.49	2.76	3.04	3.64	4.26	4.90	5.57	6.27
3.00	0.847	0.959	1.10	1.27	1.45	1.66	1.87	2.10	2.35	2.60	2.87	3.44	4.04	4.65	5.29	5.97

Table 8–6 (continued)

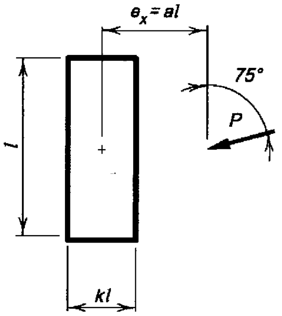
Coefficients *C* for Eccentrically Loaded Weld Groups

Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.45	5.84	6.22	6.61	6.99	7.37	7.76	8.14	8.53	8.91	9.30	10.1	10.8	11.6	12.4	13.1
0.100	5.17	5.56	5.98	6.42	6.85	7.27	7.68	8.09	8.48	8.88	9.27	10.0	10.8	11.6	12.4	13.1
0.150	5.01	5.38	5.81	6.26	6.71	7.15	7.58	8.00	8.41	8.81	9.21	10.0	10.8	11.6	12.3	13.1
0.200	4.85	5.22	5.64	6.10	6.56	7.02	7.46	7.90	8.32	8.73	9.14	9.94	10.7	11.5	12.3	13.1
0.250	4.71	5.07	5.48	5.94	6.41	6.88	7.34	7.78	8.22	8.64	9.06	9.88	10.7	11.5	12.3	13.0
0.300	4.57	4.93	5.34	5.79	6.26	6.74	7.21	7.66	8.11	8.54	8.97	9.80	10.6	11.4	12.2	13.0
0.400	4.32	4.68	5.07	5.51	5.99	6.47	6.95	7.42	7.88	8.33	8.76	9.63	10.5	11.3	12.1	12.9
0.500	4.09	4.45	4.83	5.27	5.74	6.23	6.72	7.20	7.67	8.13	8.58	9.44	10.3	11.1	12.0	12.8
0.600	3.88	4.23	4.62	5.04	5.51	5.99	6.49	6.98	7.46	7.93	8.40	9.28	10.1	11.0	11.8	12.6
0.700	3.69	4.02	4.41	4.84	5.29	5.77	6.26	6.76	7.25	7.74	8.21	9.12	10.0	10.8	11.7	12.5
0.800	3.50	3.83	4.21	4.64	5.08	5.55	6.05	6.54	7.04	7.53	8.02	8.95	9.85	10.7	11.5	12.4
0.900	3.34	3.65	4.03	4.45	4.89	5.35	5.84	6.34	6.84	7.33	7.83	8.78	9.70	10.6	11.4	12.3
1.00	3.18	3.49	3.86	4.27	4.71	5.16	5.64	6.13	6.63	7.13	7.63	8.60	9.54	10.4	11.3	12.1
1.20	2.90	3.19	3.54	3.94	4.37	4.82	5.27	5.75	6.25	6.75	7.25	8.24	9.20	10.1	11.0	11.9
1.40	2.65	2.93	3.27	3.65	4.07	4.50	4.95	5.41	5.89	6.38	6.88	7.88	8.86	9.82	10.7	11.7
1.60	2.44	2.70	3.03	3.39	3.79	4.21	4.65	5.10	5.56	6.04	6.53	7.52	8.51	9.49	10.4	11.4
1.80	2.25	2.50	2.81	3.16	3.54	3.95	4.38	4.81	5.26	5.73	6.20	7.18	8.17	9.15	10.1	11.1
2.00	2.09	2.33	2.62	2.96	3.32	3.72	4.13	4.55	4.99	5.44	5.90	6.86	7.84	8.82	9.79	10.8
2.20	1.95	2.17	2.45	2.77	3.13	3.50	3.90	4.31	4.73	5.17	5.62	6.56	7.52	8.50	9.47	10.4
2.40	1.82	2.04	2.31	2.61	2.95	3.31	3.69	4.09	4.50	4.92	5.36	6.27	7.22	8.18	9.15	10.1
2.60	1.71	1.92	2.17	2.46	2.78	3.13	3.50	3.89	4.29	4.70	5.12	6.01	6.93	7.88	8.85	9.81
2.80	1.61	1.81	2.05	2.33	2.64	2.97	3.33	3.70	4.09	4.48	4.90	5.76	6.66	7.59	8.55	9.51
3.00	1.52	1.71	1.94	2.21	2.51	2.83	3.17	3.53	3.90	4.29	4.69	5.52	6.40	7.32	8.26	9.21

Table 8–7

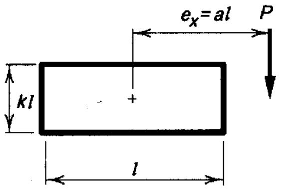
Coefficients C for Eccentrically Loaded Weld Groups

Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



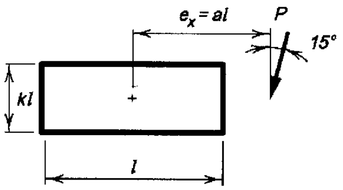
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.57	5.87	6.18	6.49	6.79	7.10	7.41	7.71	8.02	8.32	8.63	9.24	9.86	10.5	11.1	11.7
0.100	4.31	4.67	5.08	5.54	6.01	6.49	6.95	7.40	7.82	8.23	8.62	9.37	10.1	10.8	11.5	12.1
0.150	3.90	4.24	4.65	5.08	5.55	6.03	6.52	7.00	7.47	7.92	8.35	9.18	10.0	10.7	11.4	12.1
0.200	3.53	3.86	4.26	4.69	5.13	5.61	6.10	6.59	7.08	7.56	8.03	8.92	9.76	10.6	11.3	12.1
0.250	3.21	3.53	3.91	4.33	4.77	5.23	5.71	6.20	6.69	7.18	7.67	8.60	9.50	10.3	11.2	12.0
0.300	2.93	3.23	3.60	4.01	4.44	4.88	5.35	5.83	6.32	6.81	7.31	8.27	9.20	10.1	11.0	11.8
0.400	2.48	2.75	3.09	3.46	3.87	4.29	4.73	5.18	5.65	6.13	6.61	7.60	8.57	9.51	10.4	11.3
0.500	2.13	2.38	2.68	3.02	3.40	3.79	4.21	4.63	5.07	5.53	6.00	6.96	7.93	8.90	9.85	10.8
0.600	1.86	2.09	2.36	2.68	3.02	3.38	3.77	4.18	4.59	5.02	5.46	6.38	7.33	8.30	9.26	10.2
0.700	1.65	1.85	2.11	2.39	2.70	3.04	3.41	3.78	4.18	4.58	4.99	5.87	6.79	7.73	8.68	9.64
0.800	1.48	1.67	1.90	2.16	2.45	2.76	3.10	3.45	3.82	4.20	4.60	5.42	6.30	7.21	8.14	9.09
0.900	1.34	1.51	1.72	1.97	2.23	2.53	2.84	3.17	3.51	3.87	4.25	5.03	5.86	6.73	7.64	8.56
1.00	1.22	1.38	1.58	1.80	2.05	2.32	2.61	2.92	3.25	3.59	3.94	4.68	5.47	6.30	7.17	8.07
1.20	1.04	1.17	1.34	1.54	1.76	2.00	2.25	2.53	2.81	3.12	3.43	4.10	4.81	5.57	6.37	7.21
1.40	0.899	1.02	1.17	1.35	1.54	1.75	1.98	2.22	2.48	2.75	3.03	3.63	4.28	4.97	5.71	6.47
1.60	0.792	0.900	1.04	1.19	1.37	1.56	1.76	1.98	2.21	2.45	2.71	3.26	3.85	4.48	5.15	5.85
1.80	0.708	0.805	0.928	1.07	1.23	1.40	1.58	1.78	1.99	2.21	2.45	2.95	3.49	4.07	4.69	5.33
2.00	0.641	0.729	0.840	0.971	1.11	1.27	1.44	1.62	1.81	2.02	2.23	2.69	3.19	3.73	4.30	4.89
2.20	0.585	0.665	0.768	0.887	1.02	1.16	1.32	1.49	1.66	1.85	2.05	2.48	2.94	3.44	3.97	4.51
2.40	0.539	0.612	0.707	0.816	0.940	1.07	1.22	1.37	1.53	1.71	1.89	2.29	2.72	3.18	3.68	4.19
2.60	0.499	0.567	0.655	0.756	0.871	0.996	1.13	1.27	1.43	1.59	1.76	2.13	2.53	2.97	3.42	3.90
2.80	0.464	0.528	0.609	0.705	0.811	0.928	1.05	1.19	1.33	1.48	1.64	1.99	2.37	2.77	3.20	3.65
3.00	0.433	0.493	0.571	0.660	0.759	0.868	0.987	1.11	1.25	1.39	1.54	1.86	2.22	2.60	3.01	3.43

Table 8–7 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



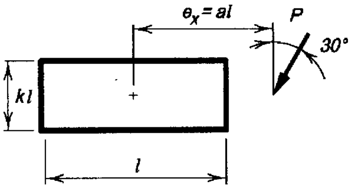
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.45	5.84	6.22	6.61	6.99	7.37	7.76	8.14	8.53	8.91	9.30	10.1	10.8	11.6	12.4	13.1
0.100	4.38	4.75	5.14	5.58	6.06	6.54	7.02	7.48	7.94	8.39	8.82	9.68	10.5	11.3	12.1	12.9
0.150	3.96	4.31	4.70	5.13	5.60	6.09	6.58	7.07	7.55	8.01	8.47	9.35	10.2	11.0	11.9	12.7
0.200	3.60	3.93	4.32	4.75	5.19	5.67	6.16	6.66	7.15	7.64	8.12	9.04	9.93	10.8	11.6	12.4
0.250	3.28	3.60	3.97	4.39	4.83	5.29	5.77	6.27	6.77	7.27	7.76	8.72	9.65	10.5	11.4	12.2
0.300	3.00	3.31	3.67	4.07	4.51	4.95	5.42	5.90	6.40	6.90	7.40	8.39	9.34	10.3	11.1	12.0
0.400	2.55	2.82	3.15	3.53	3.93	4.37	4.81	5.26	5.73	6.22	6.71	7.71	8.70	9.67	10.6	11.5
0.500	2.20	2.44	2.75	3.09	3.47	3.87	4.29	4.72	5.17	5.63	6.10	7.07	8.06	9.04	10.0	11.0
0.600	1.92	2.15	2.42	2.74	3.09	3.46	3.86	4.27	4.69	5.12	5.57	6.50	7.46	8.43	9.41	10.4
0.700	1.70	1.91	2.17	2.46	2.77	3.12	3.49	3.87	4.27	4.68	5.11	5.99	6.91	7.86	8.83	9.79
0.800	1.53	1.72	1.95	2.22	2.52	2.84	3.18	3.54	3.91	4.30	4.70	5.54	6.42	7.34	8.28	9.23
0.900	1.38	1.55	1.77	2.02	2.30	2.60	2.91	3.25	3.60	3.97	4.35	5.14	5.98	6.86	7.77	8.70
1.00	1.26	1.42	1.62	1.86	2.11	2.39	2.69	3.00	3.33	3.68	4.04	4.80	5.59	6.43	7.31	8.19
1.20	1.07	1.21	1.39	1.59	1.81	2.06	2.32	2.60	2.89	3.21	3.53	4.21	4.93	5.70	6.48	7.30
1.40	0.929	1.05	1.21	1.39	1.59	1.81	2.04	2.29	2.55	2.83	3.12	3.74	4.40	5.09	5.80	6.55
1.60	0.820	0.931	1.07	1.23	1.41	1.61	1.82	2.04	2.28	2.53	2.79	3.35	3.95	4.59	5.24	5.93
1.80	0.733	0.833	0.959	1.11	1.27	1.45	1.64	1.84	2.05	2.28	2.52	3.04	3.59	4.18	4.78	5.41
2.00	0.663	0.753	0.868	1.00	1.15	1.31	1.49	1.67	1.87	2.08	2.30	2.77	3.29	3.83	4.39	4.97
2.20	0.605	0.688	0.793	0.916	1.05	1.20	1.36	1.53	1.72	1.91	2.11	2.55	3.03	3.53	4.05	4.60
2.40	0.557	0.633	0.731	0.844	0.971	1.11	1.26	1.42	1.59	1.77	1.95	2.36	2.80	3.27	3.76	4.27
2.60	0.516	0.587	0.677	0.781	0.900	1.03	1.17	1.32	1.47	1.64	1.82	2.20	2.61	3.05	3.50	3.98
2.80	0.480	0.545	0.631	0.728	0.839	0.959	1.09	1.23	1.38	1.53	1.70	2.05	2.44	2.85	3.28	3.73
3.00	0.448	0.511	0.589	0.681	0.785	0.899	1.02	1.15	1.29	1.44	1.59	1.93	2.29	2.67	3.08	3.51

Table 8–7 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ (ϕ = 0.75, Ω = 2.00)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



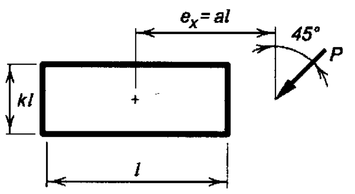
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	5.15	5.58	6.02	6.46	6.89	7.33	7.77	8.20	8.64	9.08	9.52	10.4	11.3	12.1	13.0	13.9
0.100	4.49	4.93	5.36	5.81	6.28	6.77	7.26	7.75	8.24	8.73	9.20	10.1	11.1	12.0	12.9	13.8
0.150	4.09	4.51	4.94	5.38	5.84	6.32	6.82	7.33	7.84	8.35	8.85	9.83	10.8	11.7	12.7	13.6
0.200	3.76	4.15	4.56	4.98	5.43	5.90	6.39	6.91	7.42	7.94	8.46	9.47	10.5	11.4	12.4	13.3
0.250	3.47	3.83	4.22	4.63	5.07	5.52	6.01	6.51	7.03	7.54	8.06	9.09	10.1	11.1	12.1	13.0
0.300	3.20	3.54	3.91	4.32	4.74	5.19	5.67	6.16	6.67	7.18	7.70	8.73	9.75	10.7	11.7	12.7
0.400	2.75	3.05	3.39	3.77	4.18	4.62	5.07	5.54	6.03	6.52	7.03	8.05	9.08	10.1	11.1	12.1
0.500	2.39	2.66	2.98	3.33	3.72	4.13	4.56	5.02	5.48	5.95	6.43	7.43	8.44	9.45	10.4	11.4
0.600	2.11	2.35	2.64	2.97	3.34	3.73	4.13	4.56	5.00	5.45	5.91	6.87	7.84	8.82	9.80	10.8
0.700	1.87	2.10	2.37	2.67	3.01	3.38	3.76	4.17	4.58	5.01	5.45	6.37	7.29	8.23	9.20	10.2
0.800	1.68	1.89	2.14	2.43	2.75	3.09	3.45	3.83	4.22	4.63	5.05	5.92	6.80	7.70	8.64	9.59
0.900	1.53	1.72	1.95	2.22	2.51	2.83	3.17	3.53	3.90	4.29	4.69	5.52	6.35	7.22	8.12	9.05
1.00	1.40	1.57	1.79	2.04	2.32	2.62	2.94	3.27	3.63	3.99	4.37	5.16	5.96	6.79	7.65	8.55
1.20	1.19	1.34	1.53	1.75	2.00	2.27	2.55	2.85	3.17	3.50	3.84	4.56	5.30	6.05	6.84	7.68
1.40	1.03	1.17	1.34	1.53	1.75	1.99	2.25	2.52	2.80	3.10	3.41	4.07	4.75	5.44	6.17	6.94
1.60	0.912	1.03	1.19	1.36	1.56	1.78	2.01	2.25	2.51	2.78	3.07	3.67	4.30	4.94	5.61	6.32
1.80	0.816	0.925	1.06	1.23	1.40	1.60	1.81	2.03	2.27	2.52	2.78	3.33	3.91	4.51	5.14	5.80
2.00	0.739	0.839	0.964	1.11	1.28	1.45	1.65	1.85	2.07	2.30	2.54	3.05	3.59	4.15	4.73	5.35
2.20	0.675	0.765	0.883	1.02	1.17	1.33	1.51	1.70	1.90	2.11	2.34	2.81	3.31	3.83	4.38	4.96
2.40	0.620	0.704	0.812	0.937	1.08	1.23	1.40	1.57	1.76	1.95	2.16	2.60	3.07	3.56	4.08	4.62
2.60	0.575	0.652	0.753	0.869	1.00	1.14	1.30	1.46	1.63	1.82	2.01	2.43	2.86	3.32	3.81	4.33
2.80	0.535	0.608	0.701	0.811	0.932	1.07	1.21	1.36	1.53	1.70	1.88	2.27	2.68	3.11	3.57	4.06
3.00	0.500	0.568	0.656	0.759	0.872	0.999	1.13	1.28	1.43	1.59	1.76	2.13	2.52	2.93	3.36	3.83

Table 8–7 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



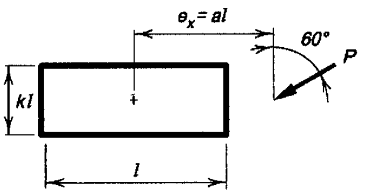
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	4.68	5.15	5.62	6.09	6.55	7.02	7.49	7.96	8.43	8.89	9.36	10.3	11.2	12.2	13.1	14.0
0.100	4.49	4.99	5.48	5.97	6.45	6.94	7.43	7.93	8.41	8.90	9.39	10.4	11.4	12.3	13.3	14.3
0.150	4.18	4.69	5.19	5.67	6.16	6.65	7.15	7.65	8.15	8.65	9.15	10.1	11.1	12.1	13.1	14.1
0.200	3.92	4.39	4.87	5.36	5.84	6.33	6.83	7.33	7.84	8.34	8.85	9.86	10.9	11.9	12.9	13.9
0.250	3.69	4.13	4.58	5.05	5.52	6.01	6.50	7.00	7.51	8.02	8.53	9.54	10.6	11.6	12.6	13.6
0.300	3.48	3.88	4.31	4.76	5.22	5.69	6.18	6.67	7.17	7.68	8.20	9.21	10.2	11.3	12.3	13.3
0.400	3.09	3.45	3.83	4.25	4.68	5.13	5.59	6.07	6.56	7.06	7.56	8.55	9.57	10.6	11.6	12.7
0.500	2.75	3.07	3.42	3.81	4.22	4.65	5.09	5.55	6.03	6.51	7.01	7.95	8.94	10.0	11.0	12.0
0.600	2.46	2.75	3.07	3.43	3.82	4.23	4.66	5.11	5.57	6.04	6.52	7.43	8.38	9.36	10.4	11.4
0.700	2.21	2.48	2.78	3.11	3.48	3.88	4.29	4.72	5.17	5.62	6.08	6.96	7.87	8.82	9.80	10.8
0.800	2.00	2.25	2.53	2.85	3.19	3.57	3.97	4.38	4.81	5.24	5.69	6.54	7.42	8.33	9.29	10.3
0.900	1.83	2.05	2.31	2.61	2.95	3.31	3.68	4.08	4.49	4.91	5.33	6.16	7.00	7.89	8.81	9.76
1.00	1.68	1.88	2.13	2.41	2.73	3.07	3.43	3.81	4.20	4.60	5.01	5.81	6.63	7.48	8.37	9.30
1.20	1.43	1.62	1.84	2.09	2.38	2.69	3.01	3.35	3.71	4.08	4.46	5.20	5.97	6.77	7.59	8.46
1.40	1.25	1.41	1.61	1.84	2.10	2.38	2.68	2.99	3.31	3.65	4.00	4.69	5.41	6.16	6.94	7.75
1.60	1.11	1.25	1.43	1.64	1.88	2.13	2.40	2.69	2.99	3.30	3.62	4.26	4.94	5.64	6.38	7.14
1.80	0.993	1.12	1.29	1.48	1.69	1.93	2.18	2.44	2.72	3.00	3.30	3.90	4.53	5.20	5.89	6.61
2.00	0.900	1.02	1.17	1.35	1.54	1.76	1.99	2.23	2.49	2.75	3.03	3.59	4.18	4.80	5.46	6.14
2.20	0.823	0.932	1.07	1.23	1.42	1.61	1.83	2.05	2.29	2.54	2.79	3.32	3.88	4.47	5.09	5.73
2.40	0.757	0.859	0.988	1.14	1.31	1.49	1.69	1.90	2.12	2.35	2.59	3.09	3.61	4.17	4.75	5.37
2.60	0.701	0.796	0.916	1.06	1.21	1.39	1.57	1.77	1.98	2.19	2.42	2.89	3.38	3.91	4.46	5.04
2.80	0.652	0.741	0.855	0.985	1.13	1.30	1.47	1.65	1.85	2.05	2.27	2.70	3.17	3.67	4.20	4.75
3.00	0.611	0.693	0.800	0.924	1.06	1.21	1.38	1.55	1.74	1.93	2.13	2.54	2.99	3.46	3.97	4.50

Table 8–7 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \frac{P_u}{\phi C_1 D l}$ $D_{min} = \frac{P_u}{\phi C C_1 l}$ $l_{min} = \frac{P_u}{\phi C C_1 D}$	$C_{min} = \frac{\Omega P_a}{C_1 D l}$ $D_{min} = \frac{\Omega P_a}{C C_1 l}$ $l_{min} = \frac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	4.37	4.89	5.40	5.91	6.43	6.94	7.46	7.97	8.49	9.00	9.52	10.5	11.6	12.6	13.6	14.7
0.100	4.26	4.79	5.31	5.82	6.34	6.86	7.37	7.89	8.40	8.92	9.44	10.5	11.5	12.6	13.6	14.7
0.150	4.13	4.67	5.20	5.71	6.22	6.73	7.25	7.75	8.26	8.77	9.29	10.3	11.3	12.4	13.5	14.5
0.200	3.97	4.51	5.05	5.57	6.08	6.58	7.08	7.59	8.09	8.59	9.10	10.1	11.1	12.2	13.2	14.3
0.250	3.85	4.36	4.87	5.39	5.90	6.40	6.90	7.40	7.89	8.39	8.89	9.90	10.9	11.9	13.0	14.0
0.300	3.74	4.22	4.72	5.22	5.72	6.21	6.70	7.19	7.68	8.17	8.67	9.67	10.7	11.7	12.7	13.7
0.400	3.50	3.94	4.40	4.87	5.36	5.84	6.32	6.78	7.25	7.73	8.21	9.19	10.2	11.2	12.2	13.3
0.500	3.26	3.66	4.09	4.53	5.00	5.47	5.94	6.40	6.86	7.31	7.78	8.72	9.70	10.7	11.7	12.7
0.600	3.02	3.38	3.78	4.21	4.65	5.11	5.57	6.03	6.48	6.93	7.38	8.30	9.25	10.2	11.2	12.2
0.700	2.79	3.13	3.51	3.91	4.33	4.77	5.22	5.68	6.12	6.56	7.00	7.91	8.83	9.78	10.8	11.8
0.800	2.58	2.90	3.25	3.63	4.04	4.46	4.90	5.35	5.79	6.22	6.65	7.54	8.45	9.38	10.3	11.3
0.900	2.39	2.69	3.02	3.38	3.77	4.18	4.61	5.04	5.48	5.90	6.33	7.20	8.09	9.01	10.0	10.9
1.00	2.23	2.50	2.82	3.16	3.53	3.93	4.34	4.77	5.19	5.61	6.02	6.88	7.76	8.66	9.59	10.5
1.20	1.94	2.19	2.47	2.78	3.13	3.49	3.88	4.28	4.69	5.09	5.48	6.30	7.15	8.02	8.91	9.83
1.40	1.71	1.93	2.19	2.47	2.79	3.13	3.50	3.88	4.26	4.64	5.02	5.80	6.61	7.44	8.31	9.19
1.60	1.53	1.73	1.96	2.22	2.52	2.84	3.18	3.54	3.90	4.25	4.61	5.35	6.13	6.93	7.76	8.62
1.80	1.38	1.56	1.77	2.02	2.30	2.60	2.92	3.25	3.59	3.92	4.26	4.97	5.70	6.47	7.27	8.09
2.00	1.25	1.42	1.61	1.85	2.11	2.38	2.69	3.00	3.31	3.63	3.95	4.62	5.33	6.06	6.83	7.63
2.20	1.15	1.30	1.48	1.70	1.94	2.21	2.49	2.78	3.08	3.37	3.68	4.32	4.99	5.69	6.43	7.19
2.40	1.06	1.20	1.37	1.57	1.80	2.05	2.31	2.59	2.87	3.15	3.44	4.05	4.69	5.36	6.07	6.81
2.60	0.981	1.11	1.27	1.46	1.68	1.91	2.16	2.42	2.69	2.95	3.23	3.81	4.42	5.07	5.74	6.45
2.80	0.915	1.04	1.19	1.37	1.57	1.79	2.03	2.27	2.53	2.78	3.04	3.59	4.18	4.79	5.45	6.12
3.00	0.857	0.972	1.12	1.29	1.47	1.68	1.91	2.14	2.38	2.62	2.87	3.40	3.95	4.55	5.18	5.83

Table 8–7 (continued)

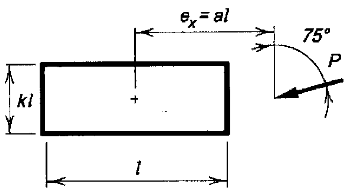
Coefficients *C* for Eccentrically Loaded Weld Groups

Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



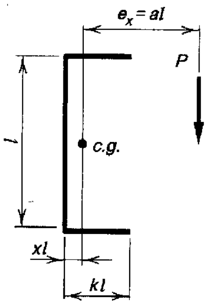
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.85	4.39	4.94	5.48	6.03	6.57	7.12	7.66	8.21	8.75	9.30	10.4	11.5	12.6	13.7	14.7
0.100	3.81	4.35	4.90	5.45	5.99	6.54	7.09	7.63	8.18	8.73	9.27	10.4	11.5	12.6	13.6	14.8
0.150	3.85	4.31	4.86	5.41	5.95	6.50	7.04	7.58	8.13	8.67	9.21	10.3	11.4	12.5	13.6	14.7
0.200	3.84	4.26	4.81	5.36	5.90	6.44	6.98	7.52	8.06	8.60	9.14	10.2	11.3	12.4	13.5	14.5
0.250	3.83	4.23	4.75	5.30	5.84	6.38	6.91	7.45	7.99	8.53	9.06	10.1	11.2	12.3	13.3	14.4
0.300	3.81	4.22	4.73	5.24	5.77	6.31	6.84	7.38	7.91	8.44	8.97	10.0	11.1	12.2	13.2	14.3
0.400	3.78	4.21	4.68	5.17	5.67	6.18	6.69	7.21	7.72	8.24	8.76	9.81	10.9	11.9	13.0	14.0
0.500	3.72	4.17	4.63	5.10	5.59	6.07	6.57	7.07	7.57	8.07	8.57	9.59	10.6	11.7	12.7	13.7
0.600	3.65	4.10	4.55	5.01	5.48	5.96	6.44	6.92	7.41	7.90	8.40	9.39	10.4	11.4	12.4	13.5
0.700	3.55	4.00	4.45	4.91	5.36	5.83	6.30	6.77	7.25	7.73	8.21	9.19	10.2	11.2	12.2	13.2
0.800	3.45	3.88	4.33	4.78	5.23	5.68	6.14	6.61	7.07	7.54	8.02	8.98	10.0	10.9	11.9	12.9
0.900	3.35	3.76	4.20	4.65	5.09	5.54	5.98	6.43	6.89	7.36	7.83	8.77	9.73	10.7	11.7	12.7
1.00	3.23	3.64	4.06	4.51	4.94	5.38	5.82	6.26	6.71	7.17	7.63	8.57	9.52	10.5	11.5	12.5
1.20	3.00	3.38	3.78	4.20	4.64	5.06	5.49	5.92	6.36	6.80	7.25	8.16	9.09	10.0	11.0	12.0
1.40	2.77	3.13	3.51	3.91	4.33	4.75	5.17	5.59	6.01	6.44	6.88	7.77	8.69	9.62	10.6	11.5
1.60	2.57	2.89	3.25	3.64	4.05	4.46	4.86	5.27	5.68	6.10	6.53	7.41	8.30	9.22	10.2	11.1
1.80	2.38	2.68	3.02	3.39	3.78	4.18	4.58	4.97	5.38	5.79	6.20	7.06	7.94	8.84	9.77	10.7
2.00	2.21	2.49	2.81	3.16	3.54	3.93	4.32	4.70	5.10	5.49	5.90	6.74	7.60	8.49	9.40	10.3
2.20	2.05	2.32	2.62	2.95	3.31	3.69	4.08	4.45	4.83	5.22	5.62	6.44	7.29	8.16	9.06	10.0
2.40	1.92	2.17	2.45	2.77	3.11	3.48	3.86	4.22	4.59	4.97	5.36	6.16	6.99	7.85	8.73	9.64
2.60	1.79	2.03	2.30	2.60	2.93	3.29	3.65	4.01	4.37	4.74	5.12	5.90	6.72	7.56	8.43	9.31
2.80	1.69	1.91	2.17	2.45	2.77	3.11	3.47	3.82	4.17	4.53	4.90	5.66	6.46	7.28	8.13	9.00
3.00	1.59	1.80	2.04	2.32	2.63	2.95	3.30	3.64	3.98	4.33	4.69	5.43	6.21	7.02	7.85	8.71

Table 8–8
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



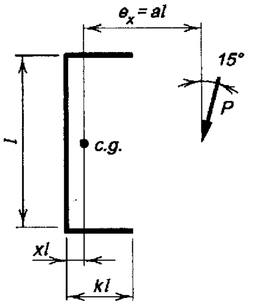
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.53	2.09	2.64	3.20	3.76	4.32	4.87	5.43	5.99	6.54	7.10	8.21	9.33	10.4	11.6	12.7
0.100	1.86	2.28	2.78	3.30	3.84	4.37	4.92	5.46	6.01	6.56	7.11	8.21	9.32	10.4	11.5	12.6
0.150	1.83	2.25	2.73	3.23	3.75	4.27	4.80	5.33	5.87	6.40	6.94	8.02	9.12	10.2	11.3	12.4
0.200	1.76	2.18	2.63	3.11	3.60	4.10	4.61	5.13	5.64	6.16	6.68	7.73	8.78	9.83	10.9	12.0
0.250	1.66	2.07	2.51	2.96	3.42	3.90	4.38	4.87	5.37	5.86	6.36	7.37	8.39	9.42	10.5	11.5
0.300	1.55	1.95	2.36	2.79	3.23	3.68	4.14	4.60	5.07	5.55	6.03	7.01	8.00	9.00	10.0	11.0
0.400	1.33	1.69	2.07	2.45	2.84	3.24	3.65	4.07	4.50	4.94	5.39	6.30	7.24	8.19	9.16	10.1
0.500	1.15	1.46	1.79	2.14	2.49	2.85	3.22	3.60	4.00	4.40	4.81	5.67	6.56	7.47	8.40	9.35
0.600	0.997	1.27	1.57	1.88	2.19	2.52	2.85	3.20	3.56	3.94	4.32	5.13	5.97	6.84	7.73	8.65
0.700	0.879	1.12	1.38	1.66	1.95	2.24	2.55	2.87	3.20	3.55	3.91	4.66	5.46	6.29	7.15	8.04
0.800	0.781	0.995	1.23	1.48	1.75	2.02	2.30	2.59	2.90	3.22	3.56	4.27	5.02	5.81	6.64	7.50
0.900	0.703	0.895	1.11	1.33	1.58	1.83	2.09	2.36	2.64	2.94	3.26	3.93	4.64	5.40	6.18	7.00
1.00	0.637	0.812	1.00	1.21	1.44	1.67	1.91	2.16	2.43	2.71	3.01	3.63	4.31	5.02	5.77	6.56
1.20	0.537	0.683	0.844	1.02	1.21	1.42	1.63	1.85	2.08	2.33	2.59	3.15	3.75	4.39	5.07	5.78
1.40	0.464	0.588	0.728	0.881	1.05	1.23	1.41	1.61	1.82	2.04	2.27	2.77	3.31	3.89	4.50	5.15
1.60	0.407	0.516	0.639	0.775	0.923	1.08	1.25	1.43	1.61	1.81	2.02	2.46	2.95	3.47	4.04	4.63
1.80	0.363	0.460	0.569	0.691	0.824	0.969	1.12	1.28	1.45	1.62	1.81	2.22	2.66	3.14	3.65	4.20
2.00	0.327	0.415	0.513	0.623	0.744	0.876	1.01	1.16	1.31	1.47	1.64	2.01	2.42	2.86	3.33	3.85
2.20	0.297	0.377	0.467	0.567	0.677	0.799	0.925	1.06	1.20	1.35	1.50	1.84	2.21	2.62	3.07	3.54
2.40	0.273	0.347	0.428	0.520	0.623	0.735	0.852	0.972	1.10	1.24	1.38	1.70	2.04	2.42	2.83	3.27
2.60	0.252	0.320	0.396	0.480	0.575	0.679	0.788	0.900	1.02	1.15	1.28	1.57	1.90	2.25	2.63	3.05
2.80	0.235	0.297	0.368	0.447	0.535	0.632	0.733	0.837	0.949	1.07	1.19	1.47	1.77	2.10	2.46	2.85
3.00	0.219	0.277	0.343	0.417	0.500	0.591	0.685	0.784	0.888	0.999	1.12	1.37	1.65	1.97	2.31	2.67
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.92	2.47	3.01	3.56	4.10	4.65	5.19	5.74	6.29	6.83	7.38	8.47	9.56	10.6	11.7	12.8
0.100	1.89	2.35	2.87	3.40	3.95	4.50	5.05	5.60	6.14	6.69	7.24	8.33	9.43	10.5	11.6	12.7
0.150	1.84	2.30	2.79	3.29	3.81	4.33	4.86	5.39	5.92	6.45	6.99	8.06	9.13	10.2	11.3	12.4
0.200	1.75	2.21	2.68	3.16	3.65	4.15	4.65	5.16	5.67	6.18	6.69	7.73	8.76	9.81	10.9	11.9
0.250	1.65	2.08	2.54	3.00	3.47	3.94	4.42	4.91	5.39	5.89	6.38	7.38	8.39	9.41	10.4	11.5
0.300	1.55	1.95	2.39	2.82	3.27	3.72	4.18	4.64	5.11	5.58	6.06	7.02	8.01	9.00	10.0	11.0
0.400	1.34	1.69	2.07	2.47	2.87	3.28	3.70	4.12	4.55	4.99	5.43	6.34	7.27	8.22	9.19	10.2
0.500	1.16	1.46	1.80	2.16	2.52	2.89	3.27	3.65	4.05	4.46	4.87	5.73	6.61	7.53	8.46	9.41
0.600	1.01	1.28	1.58	1.89	2.22	2.56	2.90	3.26	3.62	4.00	4.39	5.19	6.04	6.91	7.81	8.73
0.700	0.893	1.13	1.39	1.68	1.97	2.29	2.60	2.93	3.27	3.62	3.98	4.74	5.54	6.37	7.24	8.13
0.800	0.797	1.01	1.25	1.50	1.77	2.06	2.35	2.65	2.96	3.29	3.63	4.35	5.11	5.91	6.74	7.60
0.900	0.719	0.911	1.12	1.35	1.60	1.87	2.14	2.41	2.71	3.01	3.33	4.01	4.73	5.49	6.28	7.11
1.00	0.653	0.828	1.02	1.23	1.46	1.70	1.96	2.21	2.49	2.78	3.08	3.72	4.40	5.12	5.88	6.67
1.20	0.551	0.699	0.861	1.04	1.24	1.45	1.67	1.90	2.14	2.39	2.66	3.23	3.84	4.49	5.18	5.90
1.40	0.476	0.603	0.744	0.900	1.07	1.26	1.46	1.66	1.87	2.10	2.33	2.84	3.39	3.98	4.60	5.26
1.60	0.419	0.531	0.655	0.793	0.944	1.11	1.29	1.47	1.66	1.86	2.08	2.53	3.03	3.57	4.14	4.75
1.80	0.373	0.473	0.584	0.708	0.844	0.995	1.16	1.32	1.49	1.67	1.87	2.28	2.73	3.23	3.75	4.31
2.00	0.337	0.427	0.527	0.639	0.763	0.900	1.05	1.19	1.35	1.52	1.69	2.07	2.49	2.94	3.43	3.95
2.20	0.307	0.388	0.480	0.583	0.696	0.821	0.956	1.09	1.23	1.39	1.55	1.90	2.28	2.70	3.15	3.64
2.40	0.281	0.356	0.440	0.535	0.639	0.755	0.879	1.00	1.14	1.28	1.43	1.75	2.11	2.50	2.92	3.37
2.60	0.260	0.329	0.407	0.493	0.592	0.699	0.813	0.929	1.05	1.19	1.32	1.62	1.96	2.32	2.71	3.14
2.80	0.243	0.307	0.379	0.459	0.549	0.649	0.757	0.865	0.981	1.10	1.23	1.51	1.82	2.16	2.54	2.94
3.00	0.227	0.285	0.353	0.428	0.515	0.608	0.708	0.809	0.917	1.03	1.15	1.42	1.71	2.03	2.38	2.76
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where

P = required force, P_u or P_a, kips

D = number of sixteenths-of-an-inch in the fillet weld size

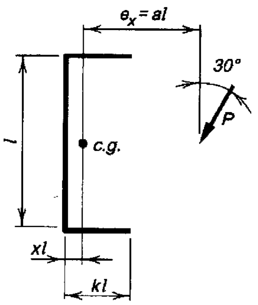
l = characteristic length of weld group, in.

a = e_x/l

e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.

C = coefficient tabulated below

C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



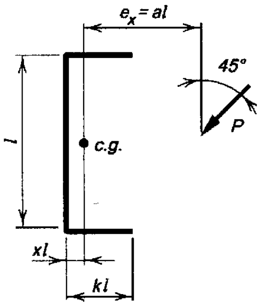
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.19	2.59	3.00	3.41	3.82	4.22	4.63	5.04	5.45	5.86	6.26	7.08	7.90	8.71	9.53	10.3
0.100	2.02	2.57	3.10	3.62	4.15	4.67	5.19	5.71	6.23	6.76	7.28	8.33	9.37	10.4	11.5	12.5
0.150	1.92	2.43	2.95	3.47	3.98	4.49	5.01	5.52	6.03	6.54	7.06	8.09	9.12	10.2	11.2	12.3
0.200	1.82	2.29	2.79	3.29	3.78	4.27	4.77	5.27	5.77	6.27	6.77	7.78	8.81	9.83	10.9	11.9
0.250	1.71	2.15	2.62	3.10	3.58	4.06	4.53	5.01	5.49	5.97	6.46	7.45	8.45	9.47	10.5	11.5
0.300	1.60	2.01	2.45	2.91	3.37	3.83	4.29	4.75	5.21	5.68	6.15	7.11	8.09	9.09	10.1	11.1
0.400	1.40	1.76	2.14	2.55	2.97	3.40	3.83	4.25	4.69	5.13	5.57	6.48	7.42	8.38	9.36	10.4
0.500	1.23	1.54	1.88	2.24	2.62	3.01	3.41	3.81	4.22	4.63	5.05	5.92	6.81	7.74	8.68	9.65
0.600	1.08	1.36	1.66	1.98	2.33	2.68	3.05	3.43	3.81	4.20	4.60	5.42	6.28	7.17	8.08	9.02
0.700	0.963	1.21	1.48	1.77	2.08	2.41	2.75	3.11	3.46	3.83	4.20	4.99	5.81	6.67	7.56	8.46
0.800	0.863	1.09	1.33	1.60	1.88	2.18	2.50	2.83	3.16	3.51	3.86	4.61	5.40	6.22	7.07	7.94
0.900	0.781	0.985	1.21	1.45	1.71	1.99	2.29	2.59	2.91	3.23	3.57	4.28	5.02	5.81	6.63	7.47
1.00	0.713	0.899	1.10	1.33	1.57	1.83	2.10	2.39	2.69	2.99	3.31	3.98	4.69	5.44	6.22	7.03
1.20	0.605	0.763	0.937	1.13	1.34	1.56	1.81	2.07	2.33	2.60	2.88	3.49	4.13	4.81	5.53	6.28
1.40	0.524	0.661	0.813	0.981	1.17	1.37	1.58	1.81	2.04	2.29	2.55	3.09	3.67	4.29	4.96	5.66
1.60	0.463	0.583	0.717	0.867	1.03	1.21	1.41	1.61	1.82	2.04	2.27	2.77	3.30	3.87	4.48	5.13
1.80	0.413	0.521	0.641	0.776	0.924	1.09	1.27	1.45	1.64	1.84	2.05	2.50	2.99	3.52	4.08	4.69
2.00	0.373	0.471	0.580	0.701	0.836	0.987	1.15	1.32	1.49	1.67	1.87	2.28	2.73	3.22	3.75	4.31
2.20	0.340	0.429	0.529	0.641	0.764	0.903	1.05	1.21	1.36	1.53	1.71	2.09	2.51	2.97	3.45	3.98
2.40	0.312	0.395	0.487	0.589	0.704	0.831	0.969	1.11	1.26	1.41	1.58	1.93	2.32	2.75	3.20	3.70
2.60	0.289	0.364	0.449	0.545	0.652	0.769	0.899	1.03	1.17	1.31	1.46	1.79	2.16	2.55	2.99	3.45
2.80	0.268	0.339	0.419	0.508	0.607	0.717	0.837	0.959	1.09	1.22	1.37	1.67	2.02	2.39	2.80	3.23
3.00	0.251	0.317	0.391	0.475	0.568	0.672	0.784	0.897	1.02	1.14	1.28	1.57	1.89	2.24	2.63	3.04
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



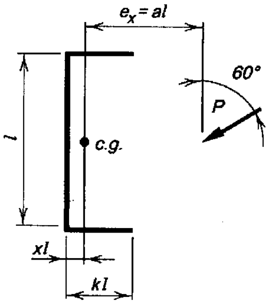
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.34	2.81	3.28	3.74	4.21	4.68	5.15	5.62	6.08	6.55	7.02	7.96	8.89	9.83	10.8	11.7
0.100	2.24	2.74	3.24	3.74	4.23	4.73	5.23	5.72	6.22	6.71	7.20	8.19	9.17	10.1	11.1	12.1
0.150	2.09	2.60	3.09	3.58	4.07	4.57	5.06	5.56	6.06	6.55	7.05	8.04	9.03	10.0	11.0	12.0
0.200	1.96	2.44	2.92	3.40	3.88	4.37	4.86	5.36	5.85	6.35	6.84	7.83	8.83	9.82	10.8	11.8
0.250	1.85	2.29	2.75	3.21	3.68	4.16	4.64	5.13	5.62	6.11	6.60	7.58	8.58	9.59	10.6	11.6
0.300	1.74	2.15	2.59	3.03	3.48	3.94	4.41	4.89	5.38	5.86	6.34	7.32	8.31	9.32	10.3	11.3
0.400	1.55	1.91	2.30	2.70	3.12	3.55	3.99	4.44	4.91	5.37	5.83	6.77	7.75	8.75	9.77	10.8
0.500	1.37	1.70	2.05	2.42	2.80	3.20	3.61	4.04	4.48	4.93	5.37	6.27	7.22	8.20	9.20	10.2
0.600	1.23	1.52	1.84	2.18	2.53	2.90	3.29	3.69	4.11	4.54	4.96	5.83	6.72	7.67	8.65	9.64
0.700	1.11	1.37	1.66	1.97	2.30	2.65	3.01	3.39	3.79	4.20	4.61	5.43	6.30	7.20	8.15	9.12
0.800	1.00	1.25	1.51	1.80	2.10	2.43	2.77	3.13	3.51	3.90	4.29	5.08	5.91	6.77	7.69	8.63
0.900	0.913	1.14	1.38	1.65	1.93	2.24	2.56	2.90	3.26	3.64	4.00	4.76	5.56	6.39	7.27	8.18
1.00	0.839	1.05	1.27	1.52	1.79	2.07	2.38	2.70	3.05	3.40	3.75	4.47	5.23	6.04	6.89	7.77
1.20	0.717	0.899	1.10	1.31	1.55	1.80	2.08	2.37	2.68	3.00	3.31	3.97	4.68	5.42	6.21	7.04
1.40	0.625	0.785	0.960	1.15	1.36	1.59	1.84	2.11	2.39	2.67	2.96	3.57	4.22	4.91	5.65	6.41
1.60	0.555	0.696	0.852	1.03	1.21	1.42	1.65	1.89	2.15	2.40	2.67	3.22	3.83	4.47	5.15	5.88
1.80	0.497	0.624	0.765	0.923	1.09	1.29	1.49	1.71	1.95	2.18	2.42	2.94	3.50	4.10	4.74	5.42
2.00	0.451	0.565	0.695	0.837	0.995	1.17	1.36	1.57	1.77	1.99	2.21	2.69	3.21	3.78	4.38	5.02
2.20	0.411	0.517	0.636	0.767	0.913	1.07	1.25	1.44	1.63	1.83	2.04	2.49	2.97	3.50	4.06	4.67
2.40	0.379	0.476	0.585	0.708	0.843	0.993	1.16	1.33	1.51	1.69	1.89	2.30	2.76	3.25	3.79	4.36
2.60	0.351	0.441	0.543	0.656	0.783	0.923	1.08	1.24	1.40	1.57	1.76	2.15	2.58	3.04	3.55	4.09
2.80	0.327	0.411	0.505	0.611	0.731	0.861	1.00	1.16	1.31	1.47	1.64	2.01	2.41	2.86	3.33	3.84
3.00	0.305	0.384	0.473	0.572	0.684	0.808	0.943	1.09	1.23	1.38	1.54	1.89	2.27	2.69	3.14	3.62
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8-8 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



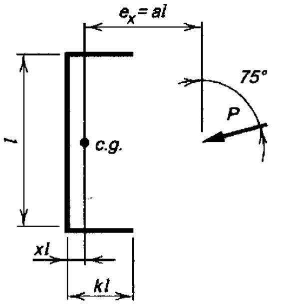
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.57	3.01	3.45	3.88	4.32	4.76	5.20	5.63	6.07	6.51	6.94	7.82	8.69	9.57	10.4	11.3
0.100	2.43	2.86	3.30	3.75	4.21	4.68	5.14	5.61	6.07	6.53	6.99	7.89	8.78	9.66	10.5	11.4
0.150	2.31	2.74	3.17	3.62	4.07	4.54	5.01	5.49	5.96	6.44	6.91	7.83	8.74	9.63	10.5	11.4
0.200	2.18	2.61	3.04	3.47	3.92	4.39	4.86	5.34	5.83	6.31	6.79	7.73	8.66	9.57	10.5	11.4
0.250	2.07	2.49	2.91	3.33	3.77	4.23	4.70	5.18	5.67	6.16	6.64	7.61	8.55	9.48	10.4	11.3
0.300	1.96	2.37	2.78	3.20	3.63	4.07	4.54	5.02	5.51	5.99	6.49	7.46	8.42	9.36	10.3	11.2
0.400	1.79	2.16	2.54	2.94	3.35	3.77	4.22	4.69	5.17	5.66	6.15	7.14	8.12	9.09	10.0	11.0
0.500	1.63	1.97	2.33	2.71	3.09	3.50	3.93	4.38	4.85	5.33	5.81	6.80	7.79	8.77	9.73	10.7
0.600	1.49	1.81	2.15	2.50	2.87	3.26	3.67	4.10	4.55	5.02	5.50	6.48	7.46	8.42	9.38	10.3
0.700	1.37	1.67	1.99	2.32	2.67	3.04	3.44	3.85	4.29	4.74	5.21	6.16	7.11	8.07	9.03	10.0
0.800	1.26	1.54	1.84	2.16	2.49	2.85	3.23	3.63	4.05	4.48	4.93	5.85	6.78	7.73	8.69	9.65
0.900	1.17	1.43	1.71	2.01	2.33	2.67	3.04	3.43	3.83	4.24	4.68	5.56	6.47	7.40	8.35	9.31
1.00	1.08	1.33	1.60	1.89	2.19	2.52	2.87	3.24	3.63	4.03	4.45	5.30	6.17	7.09	8.02	8.97
1.20	0.945	1.17	1.41	1.67	1.95	2.25	2.57	2.92	3.28	3.65	4.04	4.82	5.65	6.52	7.42	8.34
1.40	0.835	1.03	1.25	1.49	1.75	2.03	2.33	2.65	2.98	3.32	3.68	4.42	5.18	6.01	6.87	7.76
1.60	0.747	0.928	1.13	1.34	1.58	1.84	2.12	2.41	2.72	3.05	3.38	4.07	4.79	5.56	6.38	7.24
1.80	0.675	0.840	1.02	1.22	1.44	1.68	1.94	2.22	2.51	2.81	3.12	3.76	4.44	5.17	5.95	6.76
2.00	0.615	0.767	0.935	1.12	1.32	1.55	1.79	2.05	2.32	2.60	2.89	3.49	4.14	4.83	5.56	6.34
2.20	0.564	0.704	0.860	1.03	1.22	1.43	1.66	1.90	2.15	2.42	2.69	3.26	3.87	4.52	5.22	5.96
2.40	0.521	0.652	0.796	0.957	1.13	1.33	1.54	1.77	2.01	2.26	2.51	3.05	3.63	4.25	4.91	5.61
2.60	0.484	0.605	0.741	0.892	1.06	1.24	1.44	1.66	1.88	2.12	2.36	2.86	3.41	4.01	4.64	5.31
2.80	0.452	0.565	0.692	0.833	0.992	1.17	1.35	1.56	1.77	1.99	2.21	2.70	3.22	3.79	4.39	5.03
3.00	0.423	0.531	0.651	0.783	0.933	1.10	1.27	1.47	1.67	1.88	2.09	2.55	3.05	3.59	4.16	4.78
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–8 (continued)
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



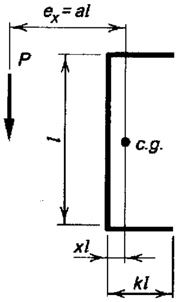
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.73	3.11	3.49	3.88	4.26	4.65	5.03	5.42	5.80	6.19	6.57	7.34	8.11	8.88	9.65	10.4
0.100	2.59	2.96	3.35	3.75	4.17	4.58	4.99	5.40	5.80	6.20	6.59	7.37	8.14	8.91	9.68	10.4
0.150	2.50	2.87	3.26	3.67	4.09	4.51	4.94	5.35	5.76	6.17	6.57	7.36	8.13	8.91	9.68	10.4
0.200	2.43	2.79	3.18	3.59	4.01	4.44	4.87	5.29	5.71	6.13	6.53	7.33	8.12	8.90	9.67	10.4
0.250	2.35	2.72	3.10	3.51	3.93	4.36	4.80	5.23	5.65	6.07	6.49	7.30	8.09	8.88	9.65	10.4
0.300	2.28	2.65	3.02	3.43	3.85	4.28	4.72	5.16	5.59	6.02	6.44	7.26	8.06	8.85	9.63	10.4
0.400	2.16	2.52	2.88	3.27	3.69	4.12	4.56	5.01	5.45	5.88	6.31	7.15	7.97	8.78	9.57	10.4
0.500	2.05	2.40	2.75	3.13	3.54	3.97	4.41	4.85	5.30	5.74	6.18	7.03	7.86	8.68	9.48	10.3
0.600	1.94	2.28	2.63	3.00	3.39	3.82	4.26	4.71	5.16	5.61	6.05	6.92	7.77	8.59	9.39	10.2
0.700	1.84	2.17	2.52	2.87	3.26	3.68	4.11	4.56	5.01	5.47	5.92	6.81	7.67	8.50	9.31	10.1
0.800	1.75	2.07	2.41	2.76	3.14	3.54	3.97	4.42	4.87	5.33	5.78	6.68	7.56	8.41	9.24	10.1
0.900	1.67	1.98	2.31	2.65	3.02	3.42	3.84	4.27	4.72	5.18	5.64	6.55	7.45	8.31	9.15	10.0
1.00	1.59	1.89	2.21	2.55	2.91	3.30	3.71	4.14	4.59	5.04	5.50	6.42	7.33	8.21	9.07	9.90
1.20	1.45	1.73	2.04	2.36	2.71	3.08	3.47	3.88	4.32	4.77	5.22	6.14	7.06	7.97	8.85	9.71
1.40	1.33	1.60	1.89	2.19	2.53	2.88	3.25	3.65	4.07	4.50	4.95	5.87	6.79	7.71	8.62	9.51
1.60	1.22	1.48	1.75	2.05	2.36	2.70	3.06	3.44	3.84	4.26	4.70	5.60	6.52	7.44	8.35	9.26
1.80	1.13	1.37	1.63	1.91	2.22	2.54	2.89	3.25	3.63	4.04	4.46	5.34	6.24	7.17	8.09	9.01
2.00	1.05	1.27	1.52	1.79	2.08	2.40	2.73	3.07	3.44	3.84	4.24	5.09	5.99	6.90	7.81	8.73
2.20	0.973	1.19	1.43	1.68	1.96	2.26	2.58	2.92	3.27	3.65	4.04	4.86	5.74	6.62	7.53	8.44
2.40	0.911	1.12	1.34	1.59	1.86	2.14	2.45	2.77	3.11	3.47	3.85	4.65	5.49	6.35	7.24	8.15
2.60	0.855	1.05	1.27	1.50	1.76	2.03	2.33	2.64	2.96	3.31	3.67	4.45	5.26	6.10	6.97	7.87
2.80	0.805	0.992	1.20	1.42	1.67	1.93	2.22	2.51	2.83	3.16	3.51	4.27	5.04	5.86	6.71	7.60
3.00	0.760	0.939	1.13	1.35	1.59	1.84	2.11	2.40	2.71	3.03	3.36	4.09	4.84	5.63	6.47	7.34
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



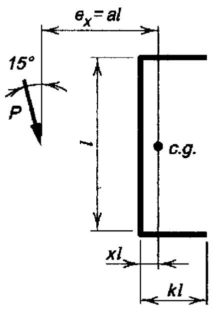
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.53	2.09	2.64	3.20	3.76	4.32	4.87	5.43	5.99	6.54	7.10	8.21	9.33	10.4	11.6	12.7
0.100	1.86	2.30	2.80	3.30	3.81	4.32	4.83	5.34	5.84	6.34	6.84	7.84	8.84	9.83	10.8	11.8
0.150	1.83	2.26	2.73	3.21	3.69	4.18	4.66	5.14	5.62	6.10	6.58	7.54	8.51	9.48	10.5	11.4
0.200	1.76	2.18	2.62	3.07	3.53	3.99	4.45	4.91	5.37	5.83	6.30	7.22	8.16	9.11	10.1	11.0
0.250	1.66	2.06	2.48	2.91	3.35	3.79	4.23	4.67	5.11	5.55	6.00	6.90	7.81	8.73	9.67	10.6
0.300	1.55	1.93	2.33	2.74	3.15	3.57	3.99	4.41	4.84	5.27	5.70	6.57	7.46	8.37	9.28	10.2
0.400	1.33	1.67	2.03	2.39	2.77	3.15	3.53	3.92	4.32	4.71	5.12	5.94	6.79	7.65	8.54	9.44
0.500	1.15	1.44	1.75	2.07	2.41	2.76	3.11	3.47	3.83	4.21	4.59	5.36	6.17	7.00	7.85	8.73
0.600	0.997	1.26	1.52	1.81	2.11	2.43	2.77	3.10	3.44	3.79	4.14	4.88	5.64	6.44	7.26	8.11
0.700	0.879	1.10	1.34	1.60	1.88	2.17	2.48	2.80	3.11	3.44	3.78	4.47	5.20	5.95	6.75	7.56
0.800	0.781	0.980	1.19	1.43	1.69	1.96	2.25	2.55	2.84	3.15	3.46	4.12	4.81	5.53	6.29	7.07
0.900	0.703	0.881	1.08	1.29	1.53	1.78	2.05	2.33	2.61	2.90	3.20	3.81	4.47	5.16	5.88	6.63
1.00	0.637	0.799	0.979	1.18	1.40	1.63	1.88	2.14	2.41	2.69	2.96	3.55	4.17	4.83	5.52	6.24
1.20	0.537	0.673	0.828	1.00	1.19	1.40	1.61	1.84	2.08	2.33	2.58	3.11	3.67	4.27	4.90	5.56
1.40	0.464	0.581	0.716	0.868	1.03	1.22	1.41	1.61	1.83	2.05	2.28	2.76	3.27	3.82	4.40	5.00
1.60	0.407	0.511	0.629	0.765	0.913	1.07	1.25	1.43	1.62	1.83	2.04	2.47	2.95	3.45	3.98	4.55
1.80	0.363	0.455	0.563	0.683	0.817	0.963	1.12	1.28	1.46	1.64	1.84	2.24	2.67	3.14	3.63	4.15
2.00	0.327	0.411	0.508	0.617	0.739	0.872	1.01	1.16	1.32	1.49	1.67	2.04	2.45	2.87	3.33	3.82
2.20	0.297	0.373	0.463	0.563	0.675	0.796	0.925	1.06	1.21	1.36	1.53	1.88	2.25	2.65	3.08	3.54
2.40	0.273	0.343	0.424	0.517	0.620	0.731	0.852	0.980	1.11	1.26	1.41	1.73	2.08	2.46	2.86	3.29
2.60	0.252	0.317	0.392	0.477	0.573	0.677	0.789	0.908	1.03	1.16	1.30	1.61	1.94	2.29	2.67	3.07
2.80	0.235	0.295	0.365	0.444	0.533	0.629	0.733	0.844	0.960	1.08	1.21	1.50	1.81	2.14	2.50	2.88
3.00	0.219	0.276	0.341	0.415	0.499	0.589	0.687	0.791	0.897	1.01	1.13	1.40	1.70	2.01	2.35	2.71
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.92	2.47	3.01	3.56	4.10	4.65	5.19	5.74	6.29	6.83	7.38	8.47	9.56	10.6	11.7	12.8
0.100	1.89	2.37	2.87	3.38	3.88	4.38	4.88	5.38	5.87	6.37	6.86	7.85	8.84	9.84	10.8	11.9
0.150	1.84	2.30	2.78	3.26	3.74	4.21	4.69	5.16	5.63	6.10	6.57	7.52	8.47	9.44	10.4	11.4
0.200	1.75	2.19	2.65	3.11	3.56	4.02	4.47	4.92	5.37	5.82	6.27	7.18	8.10	9.04	10.0	10.9
0.250	1.65	2.06	2.49	2.93	3.37	3.80	4.23	4.66	5.09	5.53	5.96	6.84	7.74	8.65	9.58	10.5
0.300	1.55	1.93	2.33	2.74	3.16	3.58	3.99	4.41	4.82	5.24	5.66	6.52	7.39	8.28	9.18	10.1
0.400	1.34	1.67	2.02	2.38	2.75	3.13	3.52	3.92	4.31	4.70	5.10	5.90	6.73	7.59	8.47	9.37
0.500	1.16	1.45	1.75	2.06	2.39	2.74	3.10	3.47	3.84	4.22	4.60	5.37	6.17	7.00	7.85	8.72
0.600	1.01	1.27	1.52	1.80	2.10	2.42	2.75	3.10	3.46	3.82	4.18	4.92	5.69	6.48	7.30	8.14
0.700	0.893	1.12	1.35	1.60	1.87	2.17	2.47	2.80	3.13	3.48	3.83	4.53	5.26	6.02	6.81	7.61
0.800	0.797	0.996	1.21	1.44	1.69	1.96	2.24	2.55	2.86	3.19	3.52	4.19	4.89	5.61	6.36	7.15
0.900	0.719	0.896	1.09	1.31	1.54	1.79	2.05	2.33	2.63	2.93	3.25	3.89	4.55	5.25	5.97	6.72
1.00	0.653	0.815	0.995	1.19	1.41	1.64	1.89	2.15	2.43	2.72	3.02	3.63	4.26	4.92	5.61	6.34
1.20	0.551	0.688	0.844	1.02	1.21	1.41	1.63	1.86	2.10	2.36	2.63	3.18	3.76	4.36	5.00	5.67
1.40	0.476	0.595	0.731	0.884	1.05	1.23	1.43	1.63	1.85	2.08	2.32	2.83	3.35	3.91	4.50	5.12
1.60	0.419	0.524	0.645	0.781	0.931	1.09	1.27	1.45	1.65	1.86	2.08	2.54	3.03	3.54	4.08	4.66
1.80	0.373	0.468	0.576	0.699	0.835	0.983	1.14	1.31	1.49	1.67	1.88	2.30	2.75	3.23	3.73	4.26
2.00	0.337	0.423	0.521	0.632	0.756	0.891	1.03	1.19	1.35	1.53	1.71	2.10	2.52	2.96	3.43	3.93
2.20	0.307	0.385	0.475	0.577	0.691	0.815	0.947	1.09	1.24	1.40	1.57	1.93	2.32	2.73	3.17	3.64
2.40	0.281	0.353	0.436	0.531	0.635	0.749	0.872	1.00	1.14	1.29	1.45	1.79	2.15	2.53	2.95	3.38
2.60	0.260	0.327	0.404	0.491	0.588	0.695	0.808	0.931	1.06	1.20	1.34	1.65	2.00	2.36	2.75	3.16
2.80	0.243	0.304	0.376	0.457	0.547	0.647	0.753	0.867	0.988	1.12	1.25	1.54	1.87	2.21	2.58	2.97
3.00	0.227	0.284	0.351	0.427	0.512	0.604	0.704	0.812	0.925	1.04	1.17	1.44	1.75	2.08	2.42	2.80
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where

P = required force, P_u or P_a, kips

D = number of sixteenths-of-an-inch in the fillet weld size

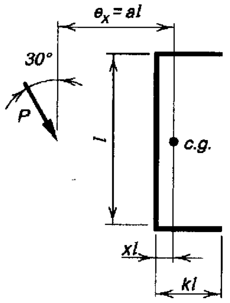
l = characteristic length of weld group, in.

a = e_x/l

e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.

C = coefficient tabulated below

C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.19	2.59	3.00	3.41	3.82	4.22	4.63	5.04	5.45	5.86	6.26	7.08	7.90	8.71	9.53	10.3
0.100	2.02	2.56	3.06	3.55	4.02	4.50	4.99	5.47	5.95	6.43	6.92	7.90	8.90	9.91	10.9	11.9
0.150	1.92	2.41	2.90	3.37	3.83	4.28	4.74	5.19	5.65	6.12	6.59	7.54	8.51	9.51	10.5	11.5
0.200	1.82	2.27	2.72	3.16	3.60	4.03	4.46	4.89	5.34	5.78	6.23	7.16	8.11	9.09	10.1	11.1
0.250	1.71	2.13	2.55	2.96	3.37	3.78	4.19	4.60	5.02	5.46	5.90	6.79	7.72	8.68	9.66	10.7
0.300	1.60	1.99	2.38	2.77	3.16	3.55	3.94	4.34	4.75	5.18	5.61	6.48	7.38	8.31	9.26	10.2
0.400	1.40	1.74	2.08	2.42	2.78	3.13	3.50	3.89	4.29	4.70	5.12	5.95	6.80	7.69	8.60	9.54
0.500	1.23	1.52	1.82	2.12	2.44	2.78	3.14	3.51	3.89	4.28	4.68	5.49	6.31	7.16	8.04	8.94
0.600	1.08	1.34	1.60	1.88	2.18	2.49	2.83	3.18	3.54	3.92	4.30	5.09	5.87	6.69	7.52	8.40
0.700	0.963	1.19	1.43	1.69	1.96	2.26	2.57	2.90	3.24	3.60	3.97	4.73	5.48	6.26	7.07	7.91
0.800	0.863	1.07	1.29	1.53	1.78	2.06	2.35	2.66	2.98	3.32	3.67	4.40	5.13	5.88	6.66	7.46
0.900	0.781	0.968	1.17	1.39	1.63	1.89	2.16	2.45	2.76	3.08	3.41	4.11	4.81	5.53	6.28	7.06
1.00	0.713	0.884	1.07	1.28	1.51	1.74	2.00	2.27	2.56	2.86	3.18	3.85	4.52	5.22	5.94	6.69
1.20	0.605	0.751	0.916	1.10	1.30	1.51	1.74	1.98	2.24	2.51	2.79	3.40	4.03	4.67	5.34	6.04
1.40	0.524	0.652	0.799	0.961	1.14	1.33	1.53	1.75	1.98	2.23	2.49	3.04	3.62	4.22	4.84	5.49
1.60	0.463	0.576	0.707	0.852	1.01	1.19	1.37	1.57	1.77	2.00	2.23	2.74	3.28	3.83	4.41	5.03
1.80	0.413	0.515	0.633	0.765	0.912	1.07	1.24	1.42	1.61	1.81	2.03	2.49	2.99	3.51	4.05	4.62
2.00	0.373	0.465	0.573	0.695	0.827	0.972	1.13	1.29	1.47	1.65	1.85	2.28	2.75	3.23	3.74	4.28
2.20	0.340	0.425	0.524	0.635	0.757	0.891	1.03	1.19	1.35	1.52	1.71	2.11	2.54	2.99	3.46	3.97
2.40	0.312	0.391	0.481	0.585	0.699	0.821	0.955	1.10	1.25	1.41	1.58	1.95	2.36	2.78	3.23	3.70
2.60	0.289	0.361	0.447	0.541	0.648	0.763	0.887	1.02	1.16	1.31	1.47	1.82	2.20	2.60	3.02	3.47
2.80	0.268	0.336	0.415	0.504	0.604	0.712	0.828	0.952	1.08	1.23	1.37	1.70	2.06	2.44	2.84	3.26
3.00	0.251	0.315	0.388	0.472	0.565	0.667	0.776	0.892	1.02	1.15	1.29	1.60	1.93	2.29	2.67	3.08
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)

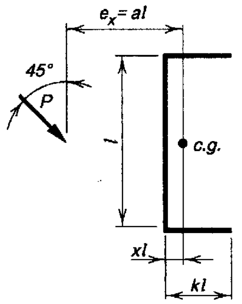
Coefficients *C* for Eccentrically Loaded Weld Groups

Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



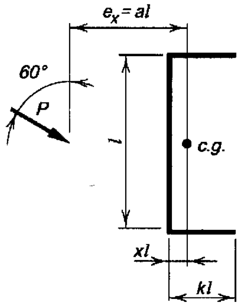
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.34	2.81	3.28	3.74	4.21	4.68	5.15	5.62	6.08	6.55	7.02	7.96	8.89	9.83	10.8	11.7
0.100	2.24	2.72	3.17	3.61	4.05	4.49	4.95	5.41	5.88	6.35	6.83	7.79	8.75	9.73	10.7	11.7
0.150	2.09	2.57	3.00	3.41	3.82	4.24	4.68	5.13	5.59	6.07	6.55	7.52	8.49	9.47	10.5	11.5
0.200	1.96	2.41	2.83	3.21	3.59	3.99	4.41	4.85	5.30	5.77	6.25	7.21	8.19	9.17	10.2	11.2
0.250	1.85	2.27	2.66	3.02	3.38	3.76	4.16	4.59	5.03	5.49	5.95	6.91	7.89	8.88	9.87	10.9
0.300	1.74	2.13	2.50	2.86	3.20	3.57	3.96	4.37	4.81	5.25	5.70	6.64	7.59	8.56	9.55	10.6
0.400	1.55	1.89	2.22	2.55	2.89	3.24	3.62	4.01	4.42	4.84	5.28	6.18	7.11	8.05	8.99	10.0
0.500	1.37	1.68	1.98	2.28	2.61	2.96	3.31	3.69	4.08	4.49	4.91	5.78	6.69	7.59	8.51	9.45
0.600	1.23	1.50	1.77	2.06	2.37	2.70	3.05	3.41	3.78	4.17	4.57	5.42	6.30	7.18	8.07	8.99
0.700	1.11	1.35	1.60	1.87	2.17	2.48	2.81	3.15	3.51	3.89	4.27	5.09	5.94	6.79	7.67	8.56
0.800	1.00	1.23	1.46	1.72	1.99	2.29	2.60	2.93	3.27	3.63	4.00	4.78	5.61	6.43	7.28	8.16
0.900	0.913	1.12	1.34	1.58	1.84	2.12	2.42	2.73	3.06	3.40	3.76	4.51	5.30	6.10	6.93	7.78
1.00	0.839	1.03	1.24	1.47	1.71	1.97	2.26	2.55	2.87	3.20	3.54	4.26	5.03	5.79	6.59	7.43
1.20	0.717	0.885	1.07	1.28	1.50	1.73	1.98	2.25	2.54	2.84	3.16	3.82	4.53	5.26	6.00	6.78
1.40	0.625	0.775	0.941	1.13	1.33	1.54	1.77	2.01	2.27	2.55	2.84	3.46	4.12	4.80	5.50	6.23
1.60	0.555	0.687	0.839	1.01	1.19	1.38	1.59	1.82	2.06	2.31	2.58	3.15	3.76	4.41	5.06	5.75
1.80	0.497	0.617	0.755	0.908	1.08	1.26	1.45	1.65	1.87	2.11	2.35	2.89	3.46	4.07	4.69	5.33
2.00	0.451	0.560	0.687	0.828	0.981	1.15	1.33	1.52	1.72	1.94	2.17	2.66	3.20	3.77	4.35	4.97
2.20	0.411	0.512	0.628	0.759	0.903	1.06	1.22	1.40	1.59	1.79	2.00	2.47	2.97	3.51	4.06	4.64
2.40	0.379	0.472	0.580	0.701	0.835	0.979	1.13	1.30	1.47	1.66	1.86	2.30	2.77	3.28	3.81	4.35
2.60	0.351	0.437	0.537	0.651	0.776	0.911	1.06	1.21	1.38	1.55	1.74	2.15	2.60	3.08	3.57	4.10
2.80	0.327	0.408	0.501	0.607	0.724	0.852	0.988	1.13	1.29	1.46	1.63	2.02	2.44	2.90	3.37	3.87
3.00	0.305	0.381	0.469	0.569	0.680	0.800	0.929	1.07	1.21	1.37	1.53	1.90	2.30	2.74	3.18	3.66
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8-9 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



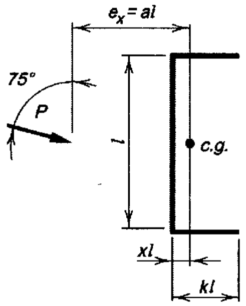
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.57	3.01	3.45	3.88	4.32	4.76	5.20	5.63	6.07	6.51	6.94	7.82	8.69	9.57	10.4	11.3
0.100	2.43	2.84	3.23	3.63	4.04	4.47	4.92	5.37	5.82	6.28	6.73	7.64	8.54	9.44	10.3	11.2
0.150	2.31	2.70	3.07	3.44	3.84	4.26	4.70	5.15	5.60	6.06	6.52	7.45	8.36	9.27	10.2	11.1
0.200	2.18	2.58	2.92	3.27	3.65	4.05	4.48	4.92	5.37	5.84	6.30	7.24	8.17	9.09	10.0	10.9
0.250	2.07	2.46	2.79	3.12	3.49	3.89	4.30	4.73	5.17	5.62	6.08	7.01	7.96	8.90	9.83	10.8
0.300	1.96	2.34	2.67	2.99	3.36	3.75	4.15	4.57	5.01	5.45	5.90	6.81	7.74	8.68	9.63	10.6
0.400	1.79	2.13	2.45	2.77	3.12	3.49	3.89	4.30	4.72	5.16	5.60	6.49	7.39	8.30	9.22	10.2
0.500	1.63	1.95	2.25	2.57	2.91	3.26	3.65	4.05	4.46	4.89	5.32	6.21	7.11	8.01	8.92	9.82
0.600	1.49	1.78	2.07	2.39	2.71	3.06	3.43	3.81	4.22	4.64	5.07	5.95	6.85	7.74	8.65	9.55
0.700	1.37	1.64	1.92	2.22	2.54	2.88	3.23	3.60	3.99	4.40	4.82	5.69	6.58	7.49	8.39	9.30
0.800	1.26	1.52	1.78	2.07	2.38	2.71	3.05	3.41	3.79	4.18	4.59	5.45	6.33	7.23	8.14	9.05
0.900	1.17	1.41	1.66	1.94	2.23	2.55	2.88	3.23	3.60	3.98	4.38	5.21	6.08	6.98	7.88	8.79
1.00	1.08	1.31	1.56	1.82	2.10	2.41	2.73	3.07	3.42	3.79	4.18	4.99	5.85	6.73	7.63	8.54
1.20	0.945	1.15	1.37	1.62	1.88	2.16	2.46	2.77	3.10	3.45	3.82	4.59	5.41	6.27	7.14	8.04
1.40	0.835	1.02	1.23	1.45	1.69	1.95	2.23	2.53	2.84	3.16	3.50	4.23	5.01	5.83	6.69	7.55
1.60	0.747	0.917	1.11	1.32	1.54	1.78	2.04	2.31	2.60	2.91	3.23	3.92	4.66	5.44	6.27	7.09
1.80	0.675	0.831	1.01	1.20	1.41	1.63	1.87	2.13	2.40	2.69	3.00	3.64	4.35	5.09	5.87	6.67
2.00	0.615	0.759	0.923	1.10	1.30	1.51	1.73	1.97	2.23	2.50	2.79	3.40	4.06	4.78	5.52	6.28
2.20	0.564	0.699	0.851	1.02	1.20	1.40	1.61	1.84	2.07	2.33	2.60	3.18	3.81	4.49	5.20	5.93
2.40	0.521	0.645	0.788	0.947	1.12	1.31	1.50	1.71	1.94	2.18	2.44	2.99	3.59	4.23	4.91	5.61
2.60	0.484	0.601	0.735	0.883	1.05	1.22	1.41	1.61	1.82	2.05	2.29	2.82	3.39	4.00	4.65	5.31
2.80	0.452	0.561	0.687	0.827	0.981	1.15	1.33	1.51	1.72	1.93	2.16	2.66	3.21	3.79	4.41	5.05
3.00	0.423	0.527	0.645	0.779	0.924	1.08	1.25	1.43	1.62	1.83	2.05	2.52	3.04	3.60	4.20	4.81
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–9 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x/l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.73	3.11	3.49	3.88	4.26	4.65	5.03	5.42	5.80	6.19	6.57	7.34	8.11	8.88	9.65	10.4
0.100	2.59	2.94	3.31	3.69	4.09	4.49	4.91	5.32	5.75	6.13	6.51	7.28	8.05	8.82	9.59	10.4
0.150	2.50	2.85	3.19	3.56	3.95	4.35	4.77	5.18	5.60	6.01	6.42	7.23	8.00	8.77	9.54	10.3
0.200	2.43	2.76	3.09	3.46	3.84	4.24	4.64	5.05	5.47	5.89	6.30	7.13	7.94	8.72	9.50	10.3
0.250	2.35	2.68	3.01	3.37	3.76	4.15	4.55	4.95	5.35	5.76	6.17	7.02	7.83	8.65	9.44	10.2
0.300	2.28	2.61	2.93	3.29	3.68	4.07	4.47	4.87	5.28	5.68	6.08	6.89	7.73	8.56	9.37	10.2
0.400	2.16	2.48	2.80	3.15	3.53	3.93	4.33	4.74	5.15	5.55	5.95	6.75	7.54	8.34	9.17	10.0
0.500	2.05	2.36	2.68	3.02	3.40	3.79	4.19	4.61	5.02	5.43	5.84	6.64	7.43	8.22	9.01	9.80
0.600	1.94	2.25	2.57	2.90	3.27	3.66	4.06	4.47	4.89	5.31	5.72	6.54	7.35	8.14	8.92	9.70
0.700	1.84	2.15	2.46	2.79	3.15	3.53	3.93	4.34	4.76	5.18	5.61	6.44	7.26	8.06	8.85	9.63
0.800	1.75	2.05	2.36	2.69	3.03	3.41	3.80	4.21	4.63	5.06	5.48	6.33	7.16	7.97	8.78	9.57
0.900	1.67	1.95	2.26	2.58	2.93	3.29	3.68	4.09	4.51	4.93	5.36	6.21	7.06	7.89	8.69	9.50
1.00	1.59	1.87	2.17	2.49	2.82	3.18	3.57	3.97	4.38	4.81	5.23	6.09	6.95	7.79	8.62	9.42
1.20	1.45	1.71	2.00	2.31	2.63	2.98	3.35	3.74	4.14	4.56	4.99	5.85	6.72	7.58	8.43	9.27
1.40	1.33	1.58	1.85	2.15	2.46	2.80	3.15	3.52	3.92	4.33	4.74	5.61	6.48	7.36	8.23	9.07
1.60	1.22	1.46	1.72	2.01	2.31	2.63	2.97	3.33	3.71	4.11	4.52	5.37	6.24	7.12	8.00	8.87
1.80	1.13	1.36	1.61	1.88	2.17	2.48	2.80	3.15	3.51	3.90	4.30	5.13	6.00	6.88	7.76	8.65
2.00	1.05	1.26	1.50	1.76	2.04	2.34	2.65	2.99	3.34	3.71	4.10	4.91	5.77	6.64	7.53	8.42
2.20	0.973	1.18	1.41	1.66	1.93	2.21	2.51	2.84	3.18	3.53	3.91	4.70	5.54	6.41	7.29	8.18
2.40	0.911	1.11	1.33	1.57	1.82	2.09	2.39	2.70	3.03	3.37	3.74	4.51	5.32	6.18	7.06	7.94
2.60	0.855	1.04	1.25	1.48	1.73	1.99	2.27	2.57	2.89	3.22	3.57	4.32	5.12	5.96	6.83	7.71
2.80	0.805	0.984	1.19	1.41	1.64	1.89	2.17	2.45	2.76	3.08	3.42	4.15	4.93	5.75	6.61	7.48
3.00	0.760	0.932	1.13	1.34	1.57	1.81	2.07	2.35	2.64	2.95	3.28	3.99	4.75	5.55	6.39	7.25
x	0.000	0.008	0.029	0.056	0.089	0.125	0.164	0.204	0.246	0.289	0.333	0.424	0.516	0.610	0.704	0.800

Table 8–10

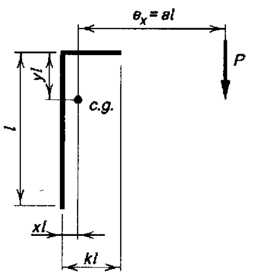
Coefficients *C* for Eccentrically Loaded Weld Groups

Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



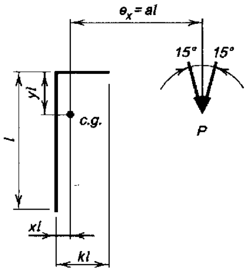
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.53	1.81	2.09	2.37	2.64	2.92	3.20	3.48	3.76	4.04	4.32	4.87	5.43	5.99	6.54	7.10
0.100	1.86	2.05	2.28	2.53	2.79	3.05	3.31	3.57	3.84	4.11	4.38	4.93	5.48	6.03	6.58	7.13
0.150	1.83	2.03	2.25	2.49	2.74	2.99	3.24	3.49	3.75	4.01	4.28	4.81	5.35	5.89	6.45	7.01
0.200	1.76	1.97	2.18	2.40	2.63	2.87	3.11	3.36	3.60	3.85	4.11	4.62	5.14	5.66	6.20	6.73
0.250	1.66	1.86	2.07	2.28	2.50	2.73	2.95	3.18	3.42	3.66	3.90	4.40	4.90	5.42	5.94	6.47
0.300	1.55	1.73	1.94	2.15	2.35	2.56	2.78	3.00	3.22	3.45	3.69	4.17	4.66	5.17	5.68	6.20
0.400	1.33	1.49	1.66	1.85	2.05	2.24	2.43	2.63	2.84	3.05	3.27	3.72	4.20	4.69	5.19	5.69
0.500	1.15	1.29	1.43	1.59	1.76	1.95	2.13	2.31	2.50	2.69	2.90	3.33	3.78	4.25	4.74	5.23
0.600	0.997	1.12	1.25	1.38	1.53	1.70	1.87	2.04	2.21	2.39	2.58	2.99	3.42	3.87	4.34	4.82
0.700	0.879	0.985	1.10	1.22	1.35	1.50	1.65	1.81	1.97	2.14	2.32	2.70	3.11	3.55	4.00	4.46
0.800	0.781	0.877	0.976	1.08	1.20	1.33	1.48	1.63	1.78	1.94	2.10	2.46	2.85	3.26	3.70	4.15
0.900	0.703	0.788	0.877	0.975	1.08	1.20	1.33	1.47	1.62	1.76	1.92	2.26	2.62	3.02	3.43	3.86
1.00	0.637	0.715	0.796	0.884	0.981	1.09	1.21	1.34	1.48	1.62	1.76	2.08	2.43	2.80	3.19	3.61
1.20	0.537	0.603	0.671	0.744	0.827	0.920	1.03	1.14	1.26	1.38	1.51	1.79	2.10	2.44	2.79	3.17
1.40	0.464	0.519	0.577	0.641	0.713	0.795	0.887	0.991	1.10	1.21	1.32	1.57	1.85	2.15	2.48	2.83
1.60	0.407	0.456	0.508	0.564	0.627	0.700	0.781	0.873	0.971	1.07	1.17	1.40	1.65	1.92	2.22	2.54
1.80	0.363	0.407	0.452	0.503	0.559	0.624	0.699	0.780	0.869	0.956	1.05	1.25	1.48	1.74	2.01	2.31
2.00	0.327	0.367	0.408	0.453	0.504	0.564	0.631	0.705	0.787	0.865	0.951	1.14	1.35	1.58	1.83	2.11
2.20	0.297	0.333	0.371	0.412	0.459	0.513	0.575	0.644	0.719	0.791	0.869	1.04	1.24	1.45	1.69	1.94
2.40	0.273	0.305	0.340	0.379	0.421	0.472	0.528	0.591	0.660	0.728	0.800	0.959	1.14	1.34	1.56	1.79
2.60	0.252	0.283	0.315	0.349	0.389	0.436	0.488	0.547	0.611	0.673	0.740	0.889	1.06	1.24	1.45	1.67
2.80	0.235	0.263	0.292	0.325	0.361	0.405	0.453	0.508	0.568	0.627	0.689	0.828	0.985	1.16	1.35	1.56
3.00	0.219	0.245	0.272	0.304	0.339	0.379	0.424	0.475	0.531	0.587	0.644	0.775	0.923	1.09	1.27	1.46
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8-10 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



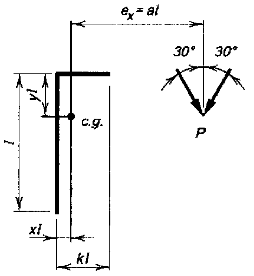
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.92	2.20	2.47	2.74	3.01	3.29	3.56	3.83	4.10	4.38	4.65	5.19	5.74	6.28	6.83	7.37
0.100	1.89	2.08	2.30	2.54	2.79	3.04	3.30	3.57	3.84	4.12	4.39	4.97	5.56	6.12	6.69	7.26
0.150	1.84	2.04	2.25	2.47	2.70	2.94	3.18	3.43	3.68	3.94	4.19	4.72	5.27	5.83	6.39	6.97
0.200	1.75	1.97	2.17	2.38	2.59	2.81	3.04	3.28	3.51	3.76	4.00	4.51	5.02	5.55	6.09	6.63
0.250	1.65	1.86	2.07	2.26	2.46	2.67	2.88	3.11	3.33	3.56	3.80	4.29	4.79	5.30	5.82	6.35
0.300	1.55	1.74	1.95	2.13	2.32	2.52	2.72	2.93	3.15	3.37	3.60	4.07	4.55	5.06	5.57	6.09
0.400	1.34	1.51	1.69	1.87	2.04	2.22	2.40	2.59	2.79	2.99	3.21	3.65	4.12	4.61	5.10	5.61
0.500	1.16	1.30	1.46	1.63	1.79	1.95	2.11	2.29	2.47	2.66	2.86	3.29	3.73	4.20	4.68	5.18
0.600	1.01	1.14	1.27	1.42	1.57	1.72	1.87	2.03	2.20	2.38	2.57	2.97	3.40	3.85	4.31	4.79
0.700	0.893	1.00	1.12	1.25	1.39	1.53	1.67	1.82	1.98	2.15	2.32	2.70	3.11	3.54	3.99	4.45
0.800	0.797	0.896	0.999	1.11	1.24	1.37	1.51	1.64	1.79	1.95	2.11	2.47	2.86	3.27	3.71	4.16
0.900	0.719	0.807	0.900	1.00	1.11	1.23	1.37	1.50	1.63	1.78	1.93	2.27	2.64	3.03	3.45	3.88
1.00	0.653	0.733	0.817	0.908	1.01	1.12	1.24	1.37	1.50	1.64	1.78	2.10	2.45	2.82	3.22	3.64
1.20	0.551	0.619	0.689	0.765	0.852	0.949	1.05	1.17	1.29	1.41	1.53	1.82	2.13	2.47	2.83	3.22
1.40	0.476	0.535	0.595	0.661	0.736	0.821	0.915	1.02	1.12	1.23	1.35	1.60	1.88	2.19	2.52	2.87
1.60	0.419	0.469	0.523	0.581	0.647	0.723	0.807	0.900	0.993	1.09	1.20	1.42	1.68	1.96	2.26	2.59
1.80	0.373	0.419	0.467	0.519	0.577	0.645	0.721	0.805	0.891	0.980	1.07	1.28	1.52	1.77	2.05	2.35
2.00	0.337	0.377	0.420	0.468	0.521	0.583	0.652	0.728	0.808	0.888	0.975	1.17	1.38	1.62	1.88	2.15
2.20	0.307	0.344	0.383	0.425	0.475	0.531	0.595	0.665	0.739	0.812	0.892	1.07	1.27	1.49	1.73	1.98
2.40	0.281	0.316	0.351	0.391	0.436	0.488	0.547	0.611	0.680	0.748	0.821	0.984	1.17	1.37	1.60	1.84
2.60	0.260	0.292	0.324	0.361	0.403	0.451	0.505	0.565	0.629	0.693	0.761	0.913	1.09	1.28	1.49	1.71
2.80	0.243	0.271	0.301	0.336	0.375	0.419	0.469	0.525	0.585	0.645	0.709	0.852	1.01	1.19	1.39	1.60
3.00	0.227	0.253	0.281	0.313	0.349	0.392	0.439	0.491	0.548	0.604	0.664	0.797	0.948	1.12	1.30	1.50
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.19	2.44	2.70	2.96	3.21	3.47	3.73	3.99	4.24	4.50	4.76	5.27	5.79	6.30	6.82	7.33
0.100	2.02	2.24	2.47	2.70	2.94	3.18	3.44	3.69	3.95	4.21	4.48	5.02	5.57	6.12	6.67	7.22
0.150	1.92	2.13	2.34	2.55	2.77	3.00	3.23	3.47	3.71	3.96	4.21	4.74	5.27	5.82	6.38	6.93
0.200	1.82	2.02	2.23	2.43	2.64	2.85	3.07	3.29	3.52	3.76	4.00	4.50	5.01	5.55	6.09	6.65
0.250	1.71	1.90	2.11	2.31	2.50	2.70	2.91	3.12	3.34	3.57	3.80	4.28	4.78	5.30	5.83	6.37
0.300	1.60	1.79	1.98	2.18	2.37	2.55	2.75	2.95	3.17	3.38	3.61	4.08	4.57	5.08	5.60	6.13
0.400	1.40	1.57	1.74	1.92	2.10	2.27	2.45	2.64	2.84	3.04	3.26	3.71	4.18	4.67	5.17	5.69
0.500	1.23	1.38	1.53	1.70	1.87	2.02	2.19	2.36	2.55	2.74	2.94	3.37	3.83	4.30	4.79	5.30
0.600	1.08	1.21	1.36	1.51	1.66	1.81	1.96	2.12	2.30	2.48	2.67	3.08	3.52	3.98	4.46	4.95
0.700	0.963	1.08	1.21	1.35	1.49	1.63	1.77	1.92	2.08	2.25	2.43	2.82	3.24	3.69	4.15	4.63
0.800	0.863	0.972	1.09	1.21	1.34	1.47	1.61	1.75	1.90	2.06	2.23	2.60	3.01	3.43	3.89	4.35
0.900	0.781	0.880	0.981	1.09	1.22	1.34	1.47	1.60	1.74	1.90	2.06	2.41	2.80	3.21	3.64	4.09
1.00	0.713	0.801	0.895	0.996	1.11	1.23	1.35	1.47	1.61	1.75	1.91	2.24	2.61	3.00	3.42	3.85
1.20	0.605	0.680	0.757	0.843	0.939	1.04	1.16	1.27	1.39	1.52	1.66	1.96	2.29	2.65	3.04	3.44
1.40	0.524	0.588	0.656	0.729	0.813	0.907	1.01	1.11	1.22	1.34	1.46	1.73	2.04	2.36	2.72	3.09
1.60	0.463	0.519	0.577	0.643	0.716	0.800	0.891	0.989	1.09	1.19	1.30	1.55	1.83	2.13	2.45	2.80
1.80	0.413	0.463	0.516	0.573	0.640	0.715	0.799	0.888	0.977	1.07	1.18	1.40	1.66	1.93	2.23	2.55
2.00	0.373	0.419	0.465	0.519	0.577	0.647	0.723	0.805	0.888	0.976	1.07	1.28	1.51	1.77	2.05	2.35
2.20	0.340	0.381	0.424	0.472	0.527	0.589	0.660	0.736	0.813	0.893	0.980	1.17	1.39	1.63	1.89	2.17
2.40	0.312	0.351	0.389	0.433	0.484	0.541	0.607	0.677	0.749	0.824	0.905	1.08	1.28	1.51	1.75	2.01
2.60	0.289	0.324	0.360	0.401	0.447	0.501	0.561	0.628	0.695	0.764	0.840	1.01	1.19	1.40	1.63	1.88
2.80	0.268	0.301	0.335	0.373	0.416	0.465	0.523	0.584	0.648	0.713	0.784	0.939	1.11	1.31	1.53	1.76
3.00	0.251	0.281	0.313	0.348	0.389	0.436	0.488	0.547	0.607	0.668	0.733	0.880	1.05	1.23	1.43	1.65
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where

P = required force, P_u or P_a, kips

D = number of sixteenths-of-an-inch in the fillet weld size

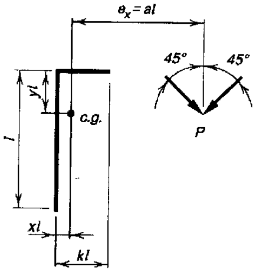
l = characteristic length of weld group, in.

$a = e_x/l$

e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.

C = coefficient tabulated below

C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.34	2.57	2.81	3.04	3.28	3.51	3.74	3.98	4.21	4.45	4.68	5.15	5.62	6.08	6.55	7.02
0.100	2.24	2.44	2.65	2.86	3.07	3.29	3.52	3.76	4.00	4.24	4.49	5.01	5.53	6.07	6.60	7.13
0.150	2.09	2.28	2.48	2.68	2.89	3.11	3.33	3.56	3.79	4.03	4.28	4.79	5.32	5.86	6.40	6.95
0.200	1.96	2.14	2.33	2.53	2.74	2.95	3.16	3.38	3.61	3.84	4.08	4.58	5.10	5.64	6.19	6.74
0.250	1.85	2.02	2.21	2.40	2.61	2.81	3.01	3.22	3.44	3.67	3.90	4.39	4.90	5.43	5.98	6.53
0.300	1.74	1.91	2.09	2.27	2.47	2.67	2.87	3.07	3.29	3.51	3.73	4.21	4.71	5.24	5.78	6.33
0.400	1.55	1.70	1.86	2.04	2.23	2.42	2.60	2.79	2.99	3.21	3.43	3.89	4.38	4.89	5.41	5.95
0.500	1.37	1.51	1.67	1.83	2.01	2.19	2.36	2.55	2.74	2.94	3.15	3.59	4.07	4.57	5.08	5.61
0.600	1.23	1.36	1.50	1.65	1.82	1.99	2.15	2.33	2.51	2.70	2.90	3.33	3.79	4.28	4.79	5.31
0.700	1.11	1.23	1.36	1.50	1.65	1.82	1.97	2.13	2.31	2.49	2.68	3.10	3.55	4.02	4.51	5.03
0.800	1.00	1.11	1.24	1.37	1.51	1.67	1.81	1.97	2.13	2.30	2.49	2.89	3.32	3.79	4.27	4.77
0.900	0.913	1.02	1.13	1.26	1.39	1.54	1.67	1.82	1.97	2.14	2.32	2.71	3.12	3.57	4.04	4.53
1.00	0.839	0.936	1.04	1.16	1.29	1.42	1.55	1.69	1.84	2.00	2.17	2.54	2.94	3.37	3.82	4.30
1.20	0.717	0.804	0.899	1.00	1.11	1.23	1.35	1.47	1.61	1.76	1.91	2.25	2.62	3.02	3.45	3.89
1.40	0.625	0.703	0.785	0.873	0.973	1.08	1.19	1.31	1.43	1.56	1.70	2.01	2.36	2.73	3.12	3.54
1.60	0.555	0.623	0.695	0.773	0.861	0.961	1.07	1.17	1.28	1.40	1.53	1.82	2.13	2.48	2.85	3.24
1.80	0.497	0.559	0.621	0.692	0.772	0.863	0.960	1.06	1.16	1.27	1.39	1.65	1.95	2.27	2.61	2.97
2.00	0.451	0.505	0.563	0.627	0.699	0.783	0.872	0.963	1.06	1.16	1.27	1.51	1.79	2.08	2.41	2.75
2.20	0.411	0.461	0.513	0.572	0.639	0.715	0.799	0.884	0.973	1.07	1.17	1.40	1.65	1.93	2.23	2.55
2.40	0.379	0.424	0.473	0.527	0.588	0.657	0.736	0.817	0.900	0.988	1.08	1.29	1.53	1.79	2.07	2.38
2.60	0.351	0.393	0.437	0.487	0.544	0.609	0.681	0.760	0.836	0.919	1.01	1.21	1.43	1.67	1.94	2.23
2.80	0.327	0.365	0.407	0.453	0.507	0.567	0.635	0.709	0.780	0.857	0.943	1.13	1.34	1.57	1.82	2.09
3.00	0.305	0.341	0.381	0.424	0.473	0.531	0.595	0.664	0.732	0.805	0.884	1.06	1.26	1.47	1.72	1.98
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)
Coefficients C
for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

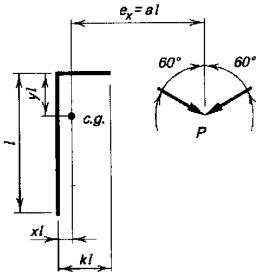
$$R_n = CC_1Dl \quad (\phi = 0.75, \ \Omega = 2.00)$$

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where

P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	3.43	3.72	4.01	4.30	4.60	4.89	5.18	5.47	5.76	6.05	6.34	6.93	7.51	6.07	6.51	6.94
0.100	3.24	3.45	3.67	3.92	4.18	4.46	4.76	5.06	5.38	5.70	6.04	6.72	7.42	6.07	6.56	7.03
0.150	3.07	3.27	3.49	3.73	3.99	4.27	4.57	4.87	5.19	5.51	5.84	6.52	7.23	5.96	6.48	6.97
0.200	2.90	3.10	3.32	3.56	3.82	4.10	4.39	4.70	5.00	5.32	5.64	6.33	7.04	5.83	6.37	6.89
0.250	2.75	2.95	3.16	3.40	3.66	3.94	4.23	4.53	4.83	5.14	5.46	6.14	6.85	5.69	6.24	6.78
0.300	2.62	2.81	3.03	3.26	3.52	3.79	4.08	4.37	4.67	4.97	5.29	5.96	6.66	5.55	6.11	6.66
0.400	2.38	2.57	2.77	3.00	3.25	3.51	3.79	4.08	4.36	4.66	4.97	5.62	6.31	5.28	5.84	6.41
0.500	2.17	2.35	2.55	2.77	3.00	3.26	3.53	3.81	4.08	4.37	4.68	5.32	5.99	5.03	5.58	6.15
0.600	1.99	2.16	2.35	2.56	2.79	3.03	3.30	3.56	3.83	4.11	4.40	5.03	5.70	4.80	5.35	5.91
0.700	1.82	1.99	2.17	2.37	2.59	2.83	3.08	3.34	3.59	3.87	4.15	4.77	5.42	4.59	5.13	5.68
0.800	1.68	1.84	2.01	2.21	2.42	2.65	2.89	3.13	3.38	3.65	3.93	4.52	5.17	4.39	4.91	5.46
0.900	1.55	1.70	1.87	2.06	2.26	2.48	2.72	2.95	3.19	3.44	3.72	4.30	4.93	4.19	4.71	5.25
1.00	1.44	1.59	1.75	1.93	2.12	2.33	2.56	2.78	3.01	3.26	3.52	4.09	4.70	4.01	4.52	5.05
1.20	1.26	1.39	1.54	1.70	1.88	2.08	2.28	2.49	2.70	2.94	3.18	3.71	4.29	3.69	4.17	4.68
1.40	1.11	1.24	1.37	1.52	1.69	1.87	2.06	2.24	2.44	2.66	2.89	3.39	3.94	3.39	3.86	4.34
1.60	0.996	1.11	1.23	1.37	1.52	1.69	1.86	2.04	2.22	2.42	2.64	3.11	3.63	3.13	3.58	4.04
1.80	0.900	1.00	1.12	1.25	1.39	1.54	1.70	1.86	2.04	2.23	2.43	2.87	3.35	2.91	3.33	3.77
2.00	0.820	0.916	1.02	1.14	1.27	1.42	1.56	1.71	1.88	2.05	2.24	2.65	3.11	2.71	3.10	3.52
2.20	0.752	0.841	0.942	1.05	1.17	1.31	1.45	1.58	1.74	1.90	2.08	2.47	2.90	2.53	2.90	3.31
2.40	0.695	0.779	0.869	0.969	1.08	1.21	1.34	1.47	1.62	1.77	1.94	2.31	2.71	2.37	2.73	3.11
2.60	0.645	0.724	0.807	0.900	1.00	1.12	1.25	1.38	1.51	1.66	1.81	2.16	2.54	2.23	2.57	2.93
2.80	0.603	0.676	0.754	0.839	0.937	1.05	1.17	1.29	1.42	1.56	1.70	2.03	2.40	2.10	2.42	2.77
3.00	0.564	0.633	0.706	0.786	0.878	0.983	1.10	1.21	1.33	1.46	1.61	1.92	2.26	1.98	2.29	2.63
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–10 (continued)

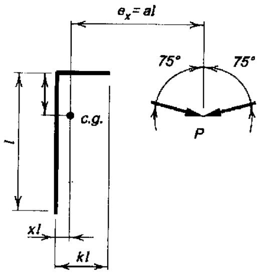
Coefficients *C*
for Eccentrically Loaded Weld Groups

Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.73	2.92	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.100	2.59	2.68	2.81	2.97	3.15	3.36	3.58	3.82	4.07	4.33	4.58	5.06	5.45	5.84	6.22	6.61
0.150	2.50	2.60	2.73	2.90	3.08	3.29	3.51	3.75	4.00	4.26	4.52	5.02	5.45	5.84	6.23	6.62
0.200	2.43	2.53	2.66	2.83	3.01	3.22	3.44	3.68	3.93	4.19	4.45	4.98	5.45	5.85	6.23	6.62
0.250	2.35	2.46	2.59	2.76	2.94	3.15	3.37	3.61	3.86	4.12	4.39	4.92	5.41	5.85	6.24	6.62
0.300	2.28	2.39	2.53	2.69	2.88	3.08	3.31	3.54	3.79	4.05	4.32	4.86	5.37	5.83	6.24	6.62
0.400	2.16	2.27	2.41	2.57	2.76	2.96	3.18	3.41	3.66	3.92	4.19	4.72	5.27	5.77	6.21	6.62
0.500	2.05	2.16	2.30	2.46	2.64	2.84	3.06	3.29	3.54	3.80	4.06	4.59	5.13	5.66	6.15	6.59
0.600	1.94	2.05	2.19	2.35	2.53	2.73	2.95	3.18	3.42	3.68	3.93	4.46	5.00	5.54	6.06	6.53
0.700	1.84	1.96	2.09	2.25	2.43	2.63	2.84	3.07	3.31	3.56	3.81	4.33	4.87	5.42	5.95	6.45
0.800	1.75	1.87	2.00	2.16	2.33	2.53	2.74	2.97	3.20	3.45	3.69	4.21	4.74	5.30	5.84	6.35
0.900	1.67	1.78	1.91	2.07	2.24	2.43	2.64	2.87	3.10	3.34	3.58	4.09	4.62	5.17	5.72	6.24
1.00	1.59	1.70	1.83	1.99	2.16	2.35	2.55	2.77	3.00	3.23	3.47	3.97	4.50	5.05	5.60	6.13
1.20	1.45	1.56	1.69	1.83	2.00	2.18	2.38	2.59	2.82	3.04	3.27	3.75	4.27	4.81	5.36	5.91
1.40	1.33	1.43	1.56	1.70	1.86	2.04	2.23	2.43	2.64	2.85	3.08	3.55	4.06	4.58	5.13	5.68
1.60	1.22	1.32	1.44	1.58	1.73	1.90	2.09	2.29	2.49	2.69	2.90	3.37	3.86	4.37	4.91	5.46
1.80	1.13	1.23	1.34	1.47	1.62	1.79	1.97	2.16	2.34	2.54	2.75	3.19	3.67	4.17	4.70	5.24
2.00	1.05	1.14	1.25	1.38	1.52	1.68	1.85	2.03	2.21	2.40	2.60	3.03	3.49	3.99	4.49	5.03
2.20	0.973	1.07	1.17	1.30	1.43	1.58	1.75	1.92	2.09	2.27	2.46	2.88	3.33	3.81	4.31	4.83
2.40	0.911	1.00	1.10	1.22	1.35	1.50	1.66	1.82	1.98	2.15	2.34	2.74	3.18	3.64	4.13	4.64
2.60	0.855	0.941	1.04	1.15	1.28	1.42	1.57	1.72	1.88	2.05	2.23	2.61	3.04	3.49	3.96	4.46
2.80	0.805	0.888	0.984	1.09	1.21	1.35	1.49	1.64	1.79	1.95	2.12	2.50	2.91	3.34	3.81	4.29
3.00	0.760	0.841	0.933	1.04	1.15	1.28	1.42	1.56	1.70	1.86	2.02	2.39	2.78	3.21	3.66	4.13
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 0°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with

$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)

or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where

P = required force, P_u or P_a, kips

D = number of sixteenths-of-an-inch in the fillet weld size

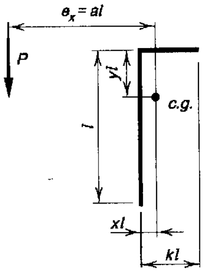
l = characteristic length of weld group, in.

a = e_x/l

e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.

C = coefficient tabulated below

C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



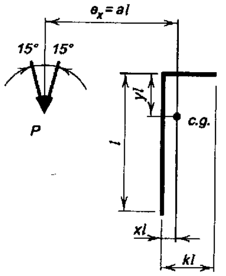
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	1.53	1.81	2.09	2.37	2.64	2.92	3.20	3.48	3.76	4.04	4.32	4.87	5.43	5.99	6.54	7.10
0.100	1.86	2.07	2.32	2.57	2.83	3.08	3.32	3.55	3.77	3.98	4.19	4.60	5.02	5.45	5.89	6.35
0.150	1.83	2.04	2.27	2.51	2.74	2.97	3.18	3.38	3.58	3.78	3.97	4.37	4.79	5.21	5.66	6.11
0.200	1.76	1.96	2.17	2.38	2.59	2.78	2.97	3.17	3.36	3.56	3.75	4.15	4.57	4.99	5.43	5.89
0.250	1.66	1.85	2.03	2.22	2.40	2.58	2.76	2.94	3.14	3.34	3.55	3.95	4.36	4.78	5.22	5.67
0.300	1.55	1.72	1.89	2.06	2.22	2.39	2.56	2.74	2.94	3.14	3.35	3.75	4.16	4.58	5.01	5.46
0.400	1.33	1.48	1.62	1.76	1.90	2.05	2.22	2.40	2.58	2.78	2.98	3.40	3.80	4.21	4.64	5.07
0.500	1.15	1.28	1.40	1.52	1.65	1.79	1.94	2.11	2.29	2.48	2.67	3.08	3.48	3.88	4.30	4.72
0.600	0.997	1.11	1.22	1.33	1.45	1.58	1.72	1.88	2.05	2.23	2.41	2.80	3.20	3.59	3.99	4.41
0.700	0.879	0.977	1.07	1.18	1.29	1.41	1.54	1.68	1.84	2.01	2.19	2.56	2.95	3.33	3.72	4.12
0.800	0.781	0.869	0.959	1.05	1.16	1.27	1.39	1.53	1.67	1.83	2.00	2.35	2.73	3.10	3.47	3.87
0.900	0.703	0.781	0.864	0.953	1.05	1.15	1.27	1.39	1.53	1.68	1.83	2.17	2.53	2.89	3.26	3.63
1.00	0.637	0.709	0.785	0.868	0.956	1.05	1.16	1.28	1.40	1.54	1.69	2.01	2.36	2.71	3.06	3.42
1.20	0.537	0.597	0.663	0.735	0.812	0.899	0.992	1.09	1.21	1.33	1.46	1.75	2.06	2.40	2.72	3.06
1.40	0.464	0.516	0.573	0.635	0.704	0.781	0.864	0.955	1.05	1.17	1.28	1.54	1.83	2.14	2.44	2.76
1.60	0.407	0.453	0.504	0.559	0.621	0.691	0.764	0.845	0.936	1.03	1.14	1.38	1.64	1.92	2.21	2.51
1.80	0.363	0.404	0.449	0.499	0.555	0.617	0.685	0.759	0.840	0.929	1.03	1.24	1.48	1.74	2.02	2.29
2.00	0.327	0.364	0.405	0.451	0.501	0.559	0.620	0.688	0.763	0.844	0.933	1.13	1.35	1.59	1.85	2.11
2.20	0.297	0.332	0.369	0.411	0.457	0.509	0.567	0.629	0.697	0.772	0.855	1.04	1.24	1.47	1.71	1.95
2.40	0.273	0.304	0.339	0.377	0.420	0.468	0.521	0.579	0.643	0.712	0.788	0.957	1.15	1.36	1.58	1.82
2.60	0.252	0.281	0.313	0.348	0.388	0.433	0.483	0.536	0.595	0.660	0.731	0.889	1.07	1.26	1.47	1.69
2.80	0.235	0.261	0.291	0.324	0.361	0.403	0.449	0.500	0.555	0.615	0.681	0.829	0.996	1.18	1.37	1.58
3.00	0.219	0.244	0.272	0.303	0.337	0.376	0.420	0.467	0.519	0.575	0.637	0.776	0.933	1.10	1.28	1.48
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 15°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x/l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



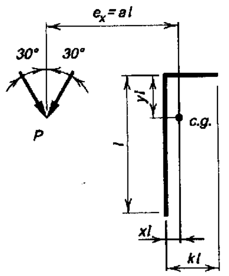
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	1.92	2.20	2.47	2.74	3.01	3.29	3.56	3.83	4.10	4.38	4.65	5.19	5.74	6.28	6.83	7.37
0.100	1.89	2.09	2.32	2.55	2.79	3.02	3.25	3.48	3.71	3.93	4.16	4.60	5.04	5.49	5.95	6.42
0.150	1.84	2.05	2.26	2.48	2.70	2.92	3.13	3.35	3.56	3.77	3.97	4.40	4.83	5.27	5.72	6.18
0.200	1.75	1.96	2.17	2.37	2.58	2.78	2.97	3.15	3.34	3.55	3.76	4.18	4.61	5.04	5.49	5.95
0.250	1.65	1.85	2.05	2.24	2.42	2.58	2.76	2.94	3.13	3.33	3.54	3.97	4.39	4.84	5.28	5.74
0.300	1.55	1.73	1.90	2.07	2.23	2.39	2.57	2.75	2.94	3.14	3.35	3.78	4.20	4.64	5.09	5.53
0.400	1.34	1.49	1.63	1.77	1.91	2.07	2.24	2.42	2.60	2.79	2.99	3.42	3.84	4.27	4.71	5.16
0.500	1.16	1.29	1.41	1.53	1.67	1.81	1.97	2.14	2.32	2.50	2.69	3.10	3.52	3.94	4.37	4.81
0.600	1.01	1.13	1.23	1.35	1.47	1.60	1.75	1.91	2.08	2.25	2.44	2.83	3.24	3.65	4.06	4.49
0.700	0.893	0.995	1.09	1.20	1.31	1.43	1.57	1.72	1.88	2.04	2.22	2.59	2.99	3.38	3.79	4.21
0.800	0.797	0.888	0.979	1.08	1.18	1.29	1.42	1.56	1.70	1.87	2.03	2.39	2.77	3.16	3.54	3.95
0.900	0.719	0.800	0.884	0.973	1.07	1.17	1.29	1.42	1.56	1.71	1.87	2.21	2.57	2.95	3.33	3.71
1.00	0.653	0.727	0.805	0.888	0.979	1.08	1.18	1.31	1.44	1.58	1.73	2.05	2.40	2.77	3.12	3.50
1.20	0.551	0.615	0.681	0.753	0.833	0.920	1.01	1.12	1.24	1.36	1.50	1.79	2.11	2.45	2.79	3.13
1.40	0.476	0.531	0.589	0.653	0.724	0.801	0.885	0.979	1.08	1.20	1.32	1.58	1.87	2.19	2.50	2.83
1.60	0.419	0.467	0.519	0.576	0.639	0.709	0.785	0.868	0.961	1.06	1.17	1.41	1.68	1.97	2.27	2.57
1.80	0.373	0.416	0.463	0.515	0.572	0.635	0.704	0.780	0.865	0.957	1.06	1.28	1.52	1.79	2.07	2.36
2.00	0.337	0.376	0.417	0.464	0.517	0.575	0.639	0.708	0.785	0.869	0.961	1.16	1.39	1.64	1.90	2.17
2.20	0.307	0.343	0.381	0.424	0.472	0.524	0.584	0.648	0.717	0.796	0.880	1.07	1.28	1.51	1.76	2.01
2.40	0.281	0.315	0.349	0.389	0.433	0.483	0.537	0.596	0.661	0.733	0.812	0.987	1.18	1.40	1.63	1.87
2.60	0.260	0.291	0.323	0.360	0.401	0.447	0.497	0.553	0.613	0.680	0.753	0.916	1.10	1.30	1.51	1.74
2.80	0.243	0.271	0.300	0.335	0.373	0.416	0.464	0.515	0.571	0.635	0.703	0.855	1.03	1.21	1.41	1.63
3.00	0.227	0.252	0.281	0.312	0.348	0.388	0.433	0.481	0.535	0.593	0.659	0.801	0.963	1.14	1.32	1.53
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients *C* for Eccentrically Loaded Weld Groups
Angle = 30°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x/l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



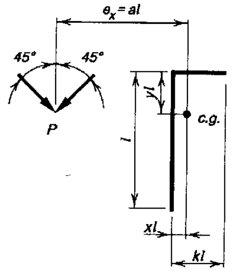
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.19	2.44	2.70	2.96	3.21	3.47	3.73	3.99	4.24	4.50	4.76	5.27	5.79	6.30	6.82	7.33
0.100	2.02	2.25	2.47	2.70	2.93	3.17	3.41	3.64	3.86	4.08	4.31	4.78	5.27	5.76	6.26	6.77
0.150	1.92	2.12	2.33	2.54	2.76	2.98	3.20	3.42	3.63	3.84	4.07	4.54	5.02	5.51	6.01	6.52
0.200	1.82	2.01	2.21	2.41	2.62	2.83	3.02	3.21	3.41	3.63	3.85	4.33	4.79	5.26	5.74	6.24
0.250	1.71	1.90	2.08	2.27	2.47	2.66	2.83	3.02	3.22	3.43	3.65	4.12	4.57	5.03	5.50	5.99
0.300	1.60	1.78	1.96	2.13	2.32	2.49	2.66	2.84	3.04	3.25	3.47	3.93	4.37	4.82	5.29	5.77
0.400	1.40	1.56	1.71	1.87	2.02	2.18	2.35	2.53	2.72	2.92	3.13	3.58	4.04	4.47	4.92	5.38
0.500	1.23	1.37	1.50	1.63	1.77	1.93	2.09	2.26	2.44	2.64	2.84	3.27	3.73	4.18	4.61	5.06
0.600	1.08	1.21	1.32	1.44	1.57	1.72	1.87	2.04	2.21	2.39	2.59	3.00	3.45	3.90	4.33	4.77
0.700	0.963	1.07	1.18	1.29	1.41	1.54	1.69	1.85	2.01	2.18	2.37	2.77	3.19	3.64	4.07	4.50
0.800	0.863	0.963	1.06	1.17	1.28	1.40	1.53	1.68	1.84	2.01	2.18	2.56	2.97	3.40	3.83	4.25
0.900	0.781	0.872	0.963	1.06	1.16	1.28	1.40	1.54	1.69	1.85	2.02	2.38	2.77	3.19	3.60	4.02
1.00	0.713	0.795	0.879	0.969	1.07	1.17	1.29	1.42	1.56	1.71	1.87	2.22	2.59	2.99	3.39	3.80
1.20	0.605	0.675	0.747	0.827	0.913	1.01	1.11	1.23	1.35	1.49	1.63	1.94	2.29	2.65	3.03	3.42
1.40	0.524	0.584	0.649	0.720	0.796	0.880	0.972	1.07	1.19	1.31	1.44	1.73	2.04	2.38	2.74	3.09
1.60	0.463	0.515	0.573	0.636	0.705	0.780	0.864	0.956	1.06	1.17	1.29	1.55	1.84	2.15	2.49	2.82
1.80	0.413	0.460	0.512	0.569	0.632	0.701	0.777	0.860	0.953	1.05	1.17	1.40	1.67	1.96	2.27	2.59
2.00	0.373	0.416	0.463	0.515	0.572	0.636	0.705	0.781	0.867	0.960	1.06	1.28	1.53	1.80	2.09	2.39
2.20	0.340	0.380	0.423	0.469	0.523	0.581	0.645	0.716	0.793	0.880	0.973	1.18	1.41	1.66	1.93	2.21
2.40	0.312	0.349	0.388	0.432	0.480	0.535	0.595	0.660	0.732	0.812	0.899	1.09	1.30	1.54	1.79	2.06
2.60	0.289	0.323	0.359	0.399	0.445	0.496	0.551	0.612	0.680	0.755	0.835	1.01	1.21	1.43	1.67	1.92
2.80	0.268	0.300	0.333	0.371	0.415	0.461	0.513	0.571	0.633	0.703	0.779	0.947	1.13	1.34	1.56	1.80
3.00	0.251	0.280	0.312	0.347	0.387	0.432	0.480	0.535	0.593	0.659	0.729	0.888	1.07	1.26	1.47	1.69
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 45°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x / l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



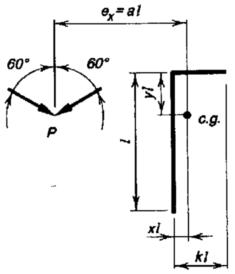
	k															
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.34	2.57	2.81	3.04	3.28	3.51	3.74	3.98	4.21	4.45	4.68	5.15	5.62	6.08	6.55	7.02
0.100	2.24	2.44	2.65	2.87	3.10	3.33	3.56	3.80	4.02	4.26	4.49	4.98	5.47	5.97	6.46	6.95
0.150	2.09	2.28	2.48	2.69	2.91	3.14	3.37	3.58	3.81	4.04	4.28	4.77	5.27	5.77	6.28	6.78
0.200	1.96	2.14	2.32	2.51	2.72	2.94	3.18	3.39	3.61	3.84	4.08	4.57	5.06	5.57	6.08	6.59
0.250	1.85	2.01	2.19	2.37	2.56	2.76	2.98	3.21	3.43	3.66	3.90	4.39	4.88	5.38	5.89	6.40
0.300	1.74	1.90	2.06	2.23	2.41	2.60	2.81	3.03	3.27	3.50	3.73	4.22	4.71	5.21	5.71	6.22
0.400	1.55	1.69	1.83	1.99	2.17	2.36	2.56	2.76	2.97	3.19	3.43	3.89	4.36	4.86	5.37	5.88
0.500	1.37	1.51	1.64	1.79	1.97	2.15	2.32	2.51	2.71	2.92	3.15	3.62	4.07	4.53	5.02	5.52
0.600	1.23	1.35	1.48	1.62	1.79	1.95	2.11	2.29	2.48	2.68	2.90	3.36	3.83	4.28	4.73	5.21
0.700	1.11	1.21	1.34	1.48	1.62	1.77	1.93	2.10	2.28	2.48	2.68	3.13	3.60	4.05	4.49	4.95
0.800	1.00	1.10	1.22	1.35	1.48	1.62	1.77	1.93	2.11	2.29	2.49	2.92	3.37	3.84	4.27	4.72
0.900	0.913	1.01	1.12	1.24	1.35	1.49	1.63	1.79	1.95	2.13	2.32	2.73	3.17	3.63	4.07	4.51
1.00	0.839	0.928	1.03	1.14	1.25	1.37	1.51	1.66	1.82	1.99	2.17	2.56	2.98	3.43	3.88	4.30
1.20	0.717	0.797	0.888	0.980	1.08	1.19	1.31	1.45	1.59	1.75	1.91	2.27	2.66	3.08	3.53	3.94
1.40	0.625	0.697	0.776	0.859	0.949	1.05	1.16	1.28	1.41	1.55	1.70	2.03	2.39	2.79	3.20	3.61
1.60	0.555	0.619	0.688	0.763	0.844	0.933	1.03	1.14	1.26	1.39	1.53	1.83	2.17	2.54	2.93	3.33
1.80	0.497	0.555	0.617	0.684	0.760	0.841	0.932	1.03	1.14	1.26	1.39	1.67	1.98	2.32	2.69	3.07
2.00	0.451	0.503	0.559	0.621	0.691	0.765	0.848	0.940	1.04	1.15	1.27	1.53	1.82	2.14	2.48	2.85
2.20	0.411	0.459	0.511	0.568	0.632	0.701	0.777	0.863	0.957	1.06	1.17	1.41	1.68	1.98	2.30	2.65
2.40	0.379	0.423	0.471	0.523	0.581	0.647	0.717	0.797	0.885	0.981	1.08	1.31	1.56	1.84	2.15	2.47
2.60	0.351	0.391	0.436	0.484	0.540	0.600	0.667	0.740	0.823	0.912	1.01	1.22	1.46	1.72	2.01	2.31
2.80	0.327	0.364	0.405	0.451	0.503	0.560	0.623	0.691	0.768	0.851	0.943	1.14	1.37	1.62	1.88	2.16
3.00	0.305	0.341	0.380	0.423	0.471	0.524	0.583	0.648	0.720	0.799	0.884	1.07	1.29	1.52	1.77	2.04
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 60°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 D l}$ $D_{min} = \dfrac{P_u}{\phi C C_1 l}$ $l_{min} = \dfrac{P_u}{\phi C C_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 D l}$ $D_{min} = \dfrac{\Omega P_a}{C C_1 l}$ $l_{min} = \dfrac{\Omega P_a}{C C_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
a = e_x / l
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



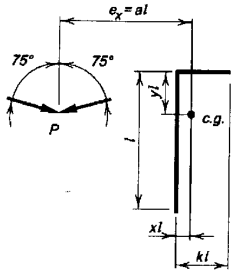
	k															
a	0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0	1.2	1.4	1.6	1.8	2.0
0.00	2.57	2.79	3.01	3.23	3.45	3.67	3.88	4.10	4.32	4.54	4.76	5.20	5.63	6.07	6.51	6.94
0.100	2.43	2.59	2.76	2.94	3.15	3.37	3.60	3.84	4.08	4.31	4.55	5.02	5.48	5.93	6.38	6.82
0.150	2.31	2.45	2.61	2.79	2.99	3.20	3.43	3.67	3.92	4.17	4.41	4.90	5.37	5.84	6.29	6.74
0.200	2.18	2.32	2.47	2.64	2.83	3.04	3.27	3.51	3.75	4.00	4.26	4.76	5.25	5.73	6.19	6.65
0.250	2.07	2.21	2.35	2.51	2.69	2.91	3.14	3.38	3.62	3.87	4.11	4.61	5.11	5.61	6.09	6.55
0.300	1.96	2.10	2.24	2.40	2.58	2.79	3.01	3.25	3.50	3.74	3.99	4.48	4.97	5.48	5.97	6.44
0.400	1.79	1.92	2.05	2.21	2.39	2.59	2.80	3.03	3.27	3.51	3.77	4.26	4.75	5.24	5.72	6.20
0.500	1.63	1.75	1.88	2.04	2.22	2.42	2.62	2.84	3.07	3.31	3.55	4.06	4.55	5.04	5.52	5.99
0.600	1.49	1.60	1.73	1.89	2.07	2.26	2.46	2.68	2.90	3.13	3.36	3.85	4.36	4.85	5.33	5.81
0.700	1.37	1.48	1.61	1.76	1.93	2.12	2.32	2.53	2.74	2.97	3.19	3.67	4.16	4.67	5.16	5.64
0.800	1.26	1.37	1.49	1.64	1.80	1.99	2.18	2.39	2.60	2.82	3.04	3.51	3.98	4.48	4.98	5.47
0.900	1.17	1.27	1.39	1.53	1.69	1.87	2.06	2.24	2.44	2.66	2.89	3.35	3.82	4.30	4.79	5.29
1.00	1.08	1.18	1.30	1.43	1.59	1.76	1.93	2.11	2.30	2.51	2.73	3.20	3.67	4.13	4.61	5.11
1.20	0.945	1.04	1.14	1.27	1.41	1.56	1.71	1.88	2.05	2.25	2.45	2.89	3.36	3.83	4.29	4.75
1.40	0.835	0.920	1.02	1.14	1.26	1.39	1.53	1.69	1.85	2.03	2.22	2.63	3.08	3.55	3.99	4.45
1.60	0.747	0.825	0.919	1.03	1.13	1.25	1.38	1.53	1.68	1.84	2.02	2.41	2.83	3.28	3.74	4.17
1.80	0.675	0.748	0.835	0.929	1.03	1.14	1.26	1.39	1.53	1.69	1.85	2.21	2.61	3.04	3.48	3.92
2.00	0.615	0.683	0.764	0.848	0.940	1.04	1.15	1.28	1.41	1.55	1.71	2.05	2.42	2.83	3.25	3.69
2.20	0.564	0.628	0.703	0.780	0.865	0.959	1.06	1.18	1.30	1.44	1.58	1.90	2.25	2.64	3.04	3.46
2.40	0.521	0.581	0.649	0.721	0.800	0.888	0.984	1.09	1.21	1.34	1.47	1.77	2.11	2.47	2.85	3.25
2.60	0.484	0.540	0.603	0.669	0.744	0.827	0.916	1.02	1.13	1.25	1.38	1.66	1.97	2.31	2.68	3.06
2.80	0.452	0.505	0.563	0.625	0.695	0.773	0.859	0.953	1.06	1.17	1.29	1.56	1.86	2.18	2.53	2.90
3.00	0.423	0.473	0.527	0.587	0.652	0.727	0.807	0.896	0.995	1.10	1.22	1.47	1.75	2.06	2.39	2.74
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

Table 8–11 (continued)
Coefficients C for Eccentrically Loaded Weld Groups
Angle = 75°

Available Strength of a weld group, ϕR_n or R_n/Ω, is determined with
$R_n = CC_1Dl$ ($\phi = 0.75$, $\Omega = 2.00$)
or

LRFD	ASD
$C_{min} = \dfrac{P_u}{\phi C_1 Dl}$ $D_{min} = \dfrac{P_u}{\phi CC_1 l}$ $l_{min} = \dfrac{P_u}{\phi CC_1 D}$	$C_{min} = \dfrac{\Omega P_a}{C_1 Dl}$ $D_{min} = \dfrac{\Omega P_a}{CC_1 l}$ $l_{min} = \dfrac{\Omega P_a}{CC_1 D}$

where
P = required force, P_u or P_a, kips
D = number of sixteenths-of-an-inch in the fillet weld size
l = characteristic length of weld group, in.
$a = e_x/l$
e_x = horizontal component of eccentricity of P with respect to centroid of weld group, in.
C = coefficient tabulated below
C_1 = electrode strength coefficient from Table 8-3 (1.0 for E70XX electrodes)



	k														
a	**0**	**0.1**	**0.2**	**0.3**	**0.4**	**0.5**	**0.6**	**0.7**	**0.8**	**0.9**	**1.0**	**1.2**	**1.4**	**1.6**	**1.8**	**2.0**
0.00	2.73	2.92	3.11	3.30	3.49	3.69	3.88	4.07	4.26	4.46	4.65	5.03	5.42	5.80	6.19	6.57
0.100	2.59	2.67	2.78	2.93	3.12	3.32	3.53	3.75	3.96	4.17	4.38	4.79	5.24	5.67	6.09	6.51
0.150	2.50	2.59	2.70	2.86	3.05	3.26	3.47	3.70	3.92	4.13	4.34	4.75	5.16	5.60	6.03	6.45
0.200	2.43	2.51	2.63	2.79	2.98	3.19	3.41	3.64	3.87	4.09	4.30	4.71	5.12	5.53	5.97	6.39
0.250	2.35	2.44	2.56	2.72	2.91	3.13	3.35	3.59	3.82	4.04	4.26	4.68	5.08	5.48	5.90	6.34
0.300	2.28	2.37	2.50	2.66	2.85	3.07	3.29	3.53	3.76	4.00	4.22	4.65	5.06	5.45	5.86	6.27
0.400	2.16	2.25	2.38	2.54	2.73	2.94	3.17	3.41	3.66	3.90	4.13	4.58	5.00	5.41	5.80	6.20
0.500	2.05	2.14	2.27	2.43	2.62	2.83	3.06	3.29	3.54	3.79	4.04	4.50	4.94	5.35	5.76	6.15
0.600	1.94	2.04	2.17	2.33	2.52	2.73	2.95	3.18	3.43	3.69	3.93	4.42	4.87	5.30	5.71	6.11
0.700	1.84	1.94	2.07	2.24	2.42	2.63	2.85	3.08	3.32	3.57	3.83	4.33	4.79	5.24	5.66	6.07
0.800	1.75	1.85	1.98	2.15	2.33	2.53	2.75	2.98	3.22	3.47	3.72	4.23	4.71	5.17	5.60	6.02
0.900	1.67	1.77	1.90	2.06	2.24	2.44	2.66	2.88	3.12	3.37	3.61	4.13	4.63	5.10	5.64	5.97
1.00	1.59	1.69	1.82	1.98	2.16	2.35	2.57	2.79	3.03	3.27	3.52	4.03	4.54	5.02	5.47	5.91
1.20	1.45	1.55	1.68	1.83	2.00	2.19	2.40	2.62	2.85	3.09	3.33	3.83	4.35	4.85	5.33	5.78
1.40	1.33	1.42	1.55	1.70	1.86	2.05	2.25	2.46	2.69	2.92	3.15	3.64	4.15	4.67	5.16	5.63
1.60	1.22	1.32	1.44	1.58	1.74	1.92	2.11	2.32	2.53	2.76	2.98	3.45	3.96	4.48	4.99	5.48
1.80	1.13	1.22	1.34	1.47	1.63	1.80	1.99	2.19	2.39	2.50	2.82	3.27	3.76	4.28	4.81	5.31
2.00	1.05	1.14	1.25	1.38	1.53	1.69	1.87	2.06	2.26	2.46	2.66	3.10	3.58	4.08	4.61	5.14
2.20	0.973	1.06	1.17	1.30	1.44	1.60	1.77	1.95	2.14	2.33	2.52	2.95	3.41	3.90	4.42	4.94
2.40	0.911	0.997	1.10	1.22	1.36	1.51	1.67	1.85	2.02	2.21	2.39	2.80	3.25	3.73	4.23	4.75
2.60	0.855	0.939	1.04	1.15	1.28	1.43	1.59	1.76	1.92	2.09	2.27	2.67	3.10	3.57	4.06	4.57
2.80	0.805	0.885	0.981	1.09	1.22	1.36	1.51	1.66	1.82	1.99	2.16	2.55	2.97	3.42	3.90	4.39
3.00	0.760	0.837	0.931	1.04	1.16	1.29	1.43	1.58	1.73	1.90	2.06	2.43	2.84	3.28	3.75	4.23
x	0.000	0.005	0.017	0.035	0.057	0.083	0.113	0.144	0.178	0.213	0.250	0.327	0.408	0.492	0.579	0.667
y	0.500	0.455	0.417	0.385	0.357	0.333	0.313	0.294	0.278	0.263	0.250	0.227	0.208	0.192	0.179	0.167

PART 9

DESIGN OF CONNECTING ELEMENTS

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of connecting elements (angles, plates, tees, gussets, etc.) used to transfer load from one structural member to another, as well as the affected elements of the connected members (beam webs, beam flanges, column webs, column flanges, etc.). For design considerations for bolts and welds, see Parts 7 and 8, respectively. For the design of connections, see Parts 10 through 15. For connecting elements that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

GROSS AREA, EFFECTIVE NET AREA, AND WHITMORE SECTION

In the determination of the available strength of connecting elements, the gross area, A_g, is of interest for the yielding limit states and the net area, A_n, is of interest for the rupture limit states. In either case, the Whitmore section may limit the effective width to less than the overall dimension of a connecting element.

Gross Area

The gross area, A_g, is determined as specified in AISC Specification Section D3.1, subject to the limitations given below for the Whitmore section.

Effective Net Area

The effective net area, A_e, is determined as specified in AISC Specification Section J4.1, subject to the limitations given below for the Whitmore section. The reduction in area for bolt holes can be determined using Table 9–1.

Whitmore Section (Effective Width)

When connecting elements are large in comparison to the bolted or welded joints within them, the Whitmore section may limit the gross and net areas of the connecting element to less than the full area (Whitmore, 1952). As illustrated in Figure 9–1, the width of the Whitmore section, l_w, is determined at the end of the joint by spreading the force from the start of the joint 30 degrees to each side in the connecting element along the line of force. The Whitmore section may spread across the joint between connecting elements, but cannot spread beyond an unconnected edge.

CONNECTING ELEMENTS SUBJECT TO COMBINED LOADING

Connection design has traditionally been based on simple stresses, such as shear, tension, compression, or flexure, not taken in combination. This simplification is adequate because connection elements are usually small or short enough that an interaction-type distribution cannot form. Even a theoretical combination analysis using the von-Mises criterion for plane stress is not any more refined. To illustrate this point, von-Mises criterion is expressed as

$$f_e = \sqrt{f_x^2 - f_x f_y + f_y^2 + 3f_{xy}^2} \le F_y$$

where

f_x and f_y = normal stresses
f_{xy} = shear stress
F_y = yield stress.

This formulation requires three stresses at any one point. Assuming f_{xy} and f_x are known for any one cut section, f_y on the perpendicular cut section is still undefined and must be assumed, thereby bringing inaccuracy into the formulation. Compounding this dilemma, f_y could be assumed as equal to zero, equal to and having the same sign as f_x, or equal to and having the opposite sign of f_x. Thus, what might appear to be a more sophisticated approach to the analysis and design of a connection does not necessarily add any reliability to the resulting design.

CONNECTING ELEMENTS SUBJECT TO SHEAR

The available strength due to shear yielding and shear rupture, ϕR_n or R_n/Ω, which must equal or exceed the required shear strength, R_u or R_a, respectively, are determined in accordance with AISC Specification Section J4.2

CONNECTING ELEMENTS SUBJECT TO TENSION

The available strength due to tension yielding and tension rupture, ϕR_n or R_n/Ω, which must equal or exceed the required tensile strength, R_u or R_a, respectively, is determined in accordance with AISC Specification Section J4.1.

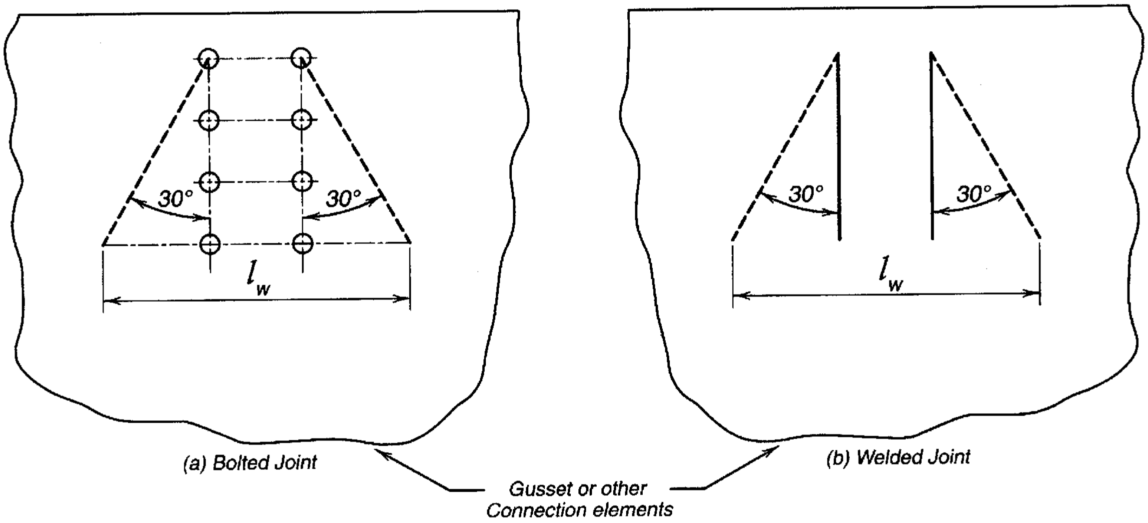


Figure 9–1. Illustration of the width of the Whitmore section.

CONNECTING ELEMENTS SUBJECT TO BLOCK SHEAR RUPTURE

The block shear available rupture strength, ϕR_n or R_n/Ω, which must equal or exceed the required strength, R_u or R_a, respectively, is determined in accordance with AISC Specification Section J4.3. The values tabulated in Table 9–3 can be used to calculate the available block shear rupture strength.

CONNECTING ELEMENT RUPTURE STRENGTH AT WELDS

In many cases, the load path from a weld to the connecting element is such that the strength of the connecting element can be evaluated directly. However, in some cases, the available strength of the connecting element is not directly calculable. For example, while the strength of the beam-web welds for a double-angle connection can be directly calculated, the strength of the beam web at this weld cannot. In cases such as these, it is often convenient to calculate the minimum base-metal thickness that will match the available shear rupture strength of the base metal to the available shear rupture strength of the weld(s).

For fillet welds with $F_{EXX} = 70$ ksi on both sides of the connecting element, the minimum thickness required to match the shear rupture strength of the connecting element to the shear rupture strength of the base metal is

$$t_{min} = \frac{0.6F_{EXX} \times \frac{\sqrt{2}}{2} \times \frac{D}{16} \times 2}{0.6F_u}$$

$$= \frac{6.19D}{F_u}$$

where

D = number of sixteenths of an inch in the weld size

F_u = specified minimum tensile strength of the connecting element, ksi

Similarly, for fillet welds with $F_{EXX} = 70$ ksi on one side of the connection, the minimum thickness required to match the shear rupture strength of the connecting element to the shear rupture strength of the base metal is

$$t_{min} = \frac{3.09D}{F_u}$$

CONNECTING ELEMENTS SUBJECT TO COMPRESSION BUCKLING

When connecting elements are subject to compression, the available strength, ϕP_n or P_n/Ω, which must equal or exceed the required compressive strength, P_u or P_a, respectively, can be determined as given in Specification Section J4.4, when $KL/r < 25$.

CONNECTING ELEMENTS SUBJECT TO FLEXURE

Connection elements are normally short enough and thick enough that flexural effects, if present at all, do not impact the design.

Yielding, Lateral-Torsional Buckling, and Local Buckling

The available flexural strength, ϕM_n or M_n/Ω, which must equal or exceed the required flexural strength, M_u or M_a, respectively, is determined in accordance with AISC Specification Chapter F. When connection elements are long enough and thin enough that flexural effects must be considered, these provisions can be applied. User Note F1.1 provides guidance based upon cross-section for which Section in Chapter F is applicable.

Treatment of coped beams is provided below based upon Cheng, et al. (1984). For beam ends with short copes no greater than the length of the connection angle(s), plate, or tee, local web buckling will generally not occur.

Rupture

For beams and rolled girders with bolt holes in the tension flange, see AISC Specification Section F13. For coped beams, see the discussion on coped beams below. In other cases for connection elements, the available flexural rupture strength, ϕM_n or M_n/Ω, can be determined as

$$M_n = F_u Z_{net}$$
$$\phi_b = 0.75 \qquad \Omega_b = 2.00$$

Coped Beams

The end reaction for a coped beam may be limited by flexural limit states such as yielding, rupture, local buckling, or lateral-torsional buckling. For a coped beam, the required flexural strength is calculated as

LRFD	ASD
$M_u = R_u e$	$M_a = R_a e$

where

R_u or R_a = beam end reaction (LRFD, ASD), kips

e = distance from the face of the cope to the point of inflection of the beam, in. It is usually assumed that the point of inflection is located at the face of the supporting member and e is as shown in Figure 9–2. However, depending upon the connection type and stiffness and support condition, the point of inflection may move away from the face of the supporting member; when this is the case, a lesser value of e may be justified. The choice of e shown in Figure 9–2 will be conservative.

For a beam coped at the top flange or both the top and bottom flanges, the available flexural rupture strength, ϕM_n or M_n/Ω, can be determined as

$$M_n = F_u S_{net}$$
$$\phi_b = 0.75 \qquad \Omega_b = 2.00$$

The available flexural local buckling strength or a beam coped at the top flange or both the top and bottom flanges must equal or exceed the required strength. The available strength, $\phi_b M_n$ or M_n/Ω_b, is determined as

$$M_n = F_{cr} S_{net}$$

$$\phi_b = 0.90 \qquad \Omega_b = 1.67$$

where

F_{cr} = available buckling stress, determined as given below, ksi
S_{net} = net section modulus, in.^{3} Values of S_{net} are tabulated in Table 9–2

When a beam is coped at the top flange only, the available buckling stress is based upon the classical plate buckling formula with a *k*-factor corresponding to the condition with three edges simply supported and one free edge. An additional factor, *f*, is applied to account for stress concentrations at the cope and to correlate the solutions with experimental results (Cheng, et. al., 1984).

The available buckling stress, ϕF_{cr} or F_{cr}/Ω, for a beam coped at the top flange only when $c \le 2d$ and $d_c \le d/2$ (see Figure 9–2) is determined as

$$F_{cr} = \frac{\pi^2 E}{12\left(1-\nu^2\right)}\left(\frac{t_w}{h_o}\right)^2 fk$$

$$= 26{,}210\left(\frac{t_w}{h_o}\right)^2 fk$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where

E = 29,000 ksi, modulus of elasticity of steel
ν = 0.3, Poisson's ratio
f = plate buckling model adjustment factor

$$= \frac{2c}{d} \text{ when } \frac{c}{d} \le 1.0$$

$$= 1 + \frac{c}{d} \text{ when } \frac{c}{d} > 1.0$$

t_w = beam web thickness
k = plate buckling coefficient

$$= 2.2\left(\frac{h_o}{c}\right)^{1.65} \text{ when } \frac{c}{h_o} \le 1.0$$

$$= \frac{2.2h_o}{c} \text{ when } \frac{c}{h_o} > 1.0$$

$h_o = d - d_c$, reduced beam depth, in. Note that, for convenience, the dimension h_o, as illustrated in Figure 9–2, is used in these calculations instead of the more correct dimension h_l to eliminate the detailed calculation required to locate the neutral axis of the coped beam. Alternatively, the dimension h_1 may be substituted for h_o in the local buckling calculations.

c = cope length as illustrated in Figure 9–2, in.
d = beam depth, in.
d_c = cope depth as illustrated in Figure 9–2, in.

When a beam is coped at both flanges, the available buckling stress, ϕF_{cr} or F_{cr}/Ω, is based upon a lateral-torsional buckling model with an adjustment factor f_d (Cheng, et al., 1984). The available buckling stress, ϕF_{cr} or F_{cr}/Ω, for a beam coped equally at both flanges with $c \leq 2d$ and $d_c \leq 0.2d$ (see Figure 9–3) is determined as

$$F_{cr} = 0.62\pi E \frac{t_w^2}{ch_o} f_d$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where

$$f_d = 3.5 - 7.5\left(\frac{d_c}{d}\right)$$

d_c = cope depth at the compression flange, in.

and all other variables are as defined previously.

When a beam is coped at both flanges and $d_c > 0.2d$, a conservative procedure also based upon the classical plate buckling equation can be used. Including both elastic and inelastic buckling, the available buckling stress, ϕF_{cr} or F_{cr}/Ω, is

$$F_{cr} = F_y Q$$
$$\phi = 0.90 \qquad \Omega = 1.67$$

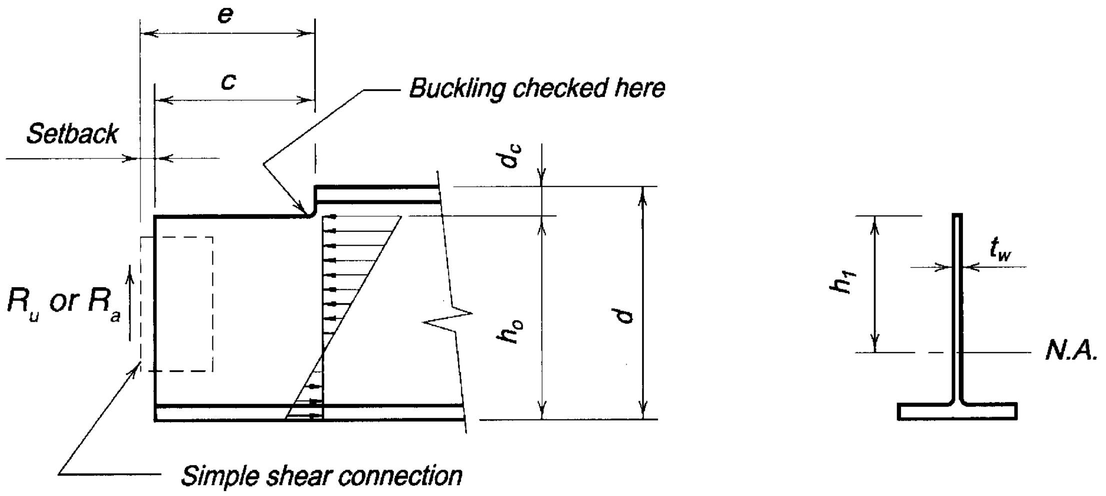


Figure 9–2. Local buckling of beam web coped at top flange only.

where

$$Q = 1 \text{ for } \lambda \le 0.7$$
$$= (1.34 - 0.486\lambda) \text{ for } 0.7 < \lambda \le 1.41$$
$$= (1.30/\lambda^2) \text{ for } \lambda > 1.41$$

$$\lambda = \frac{h_o\sqrt{F_y}}{10t_w\sqrt{475 + 280\left(\frac{h_o}{c}\right)^2}}$$

where

c = length of plate parallel to the compressive force, in.
h_o = reduced beam depth, in.
t_w = thickness of plate, in.
F_y = yield stress, ksi

BEARING LIMIT STATES

Bearing Strength at Bolt Holes

For available bearing strength at bolt holes, see Part 7.

Steel-on-Steel Bearing Strength (Other Than at Bolt Holes)

Bearing strength for applications other than at bolt holes is determined as given in AISC Specification Section J7. The fabrication and erection requirements in AISC Specification Sections M2.6, M2.8, and M4.4 are applicable to connecting elements that transfer load by contact bearing on steel.

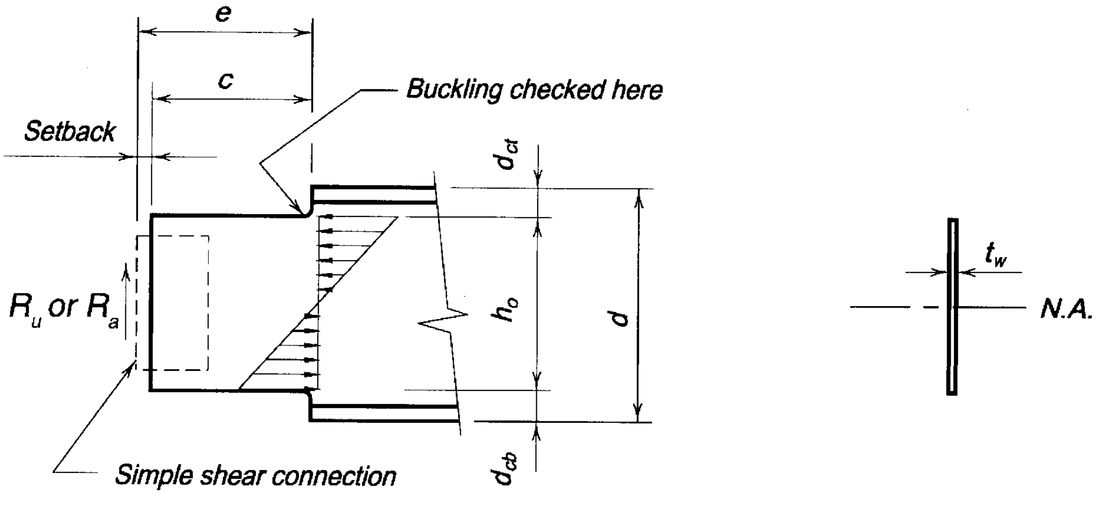


Figure 9–3. Local buckling of beam web coped at both flanges.

Bearing Strength on Concrete or Masonry

The bearing strength of concrete or masonry is determined as given in AISC Specification Section J8. The fabrication and erection requirements in AISC Specification Sections M2.8 and M4.1 are applicable to connecting elements that transfer load by contact bearing on concrete or masonry.

OTHER SPECIFICATION REQUIREMENTS AND DESIGN CONSIDERATIONS

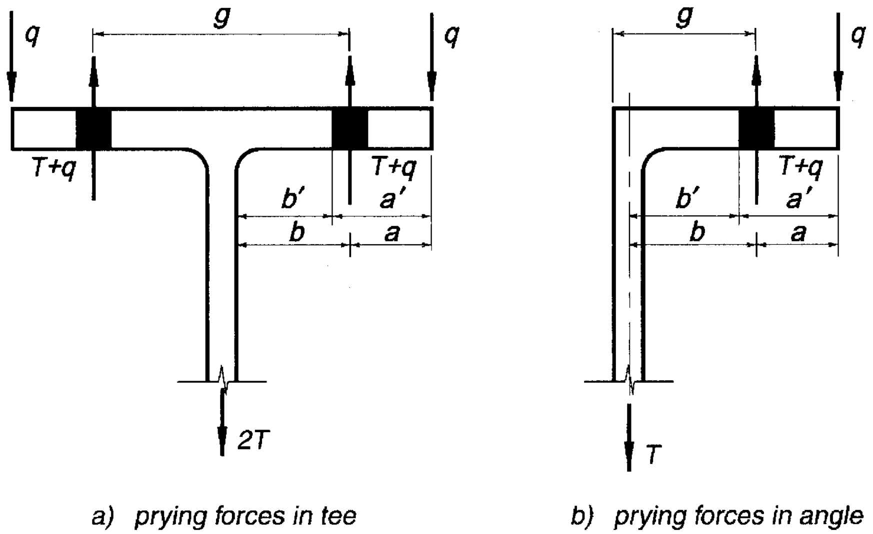
The following other specification requirements and design considerations apply to the design of connecting elements:

Prying Action

Prying action is a phenomenon (in bolted construction only and for tensile bolt forces only) whereby the deformation of a connecting element under a tensile force increases the tensile force in the bolt above that due to the direct tensile force alone. Proper design for prying action includes the selection of bolt diameter and fitting thickness such that there is sufficient stiffness and strength in the connecting element and strength in the bolt. The following discussion of prying action is similar to what has been considered in the past, except that the design basis has been changed to calculate strength in terms of F_u, which provides better correlation with available test data than previous design methods. For the development of the prying action equations presented here, see Thornton (1992) and Swanson (2002).

Consider the tee or angle used in a hanger connection, as shown in Figure 9–4. The thickness required to eliminate prying action, t_{min}, can be determined as

LRFD	ASD
$t_{min} = \sqrt{\dfrac{4.44Tb'}{pF_u}}$	$t_{min} = \sqrt{\dfrac{6.66Tb'}{pF_u}}$



a) prying forces in tee b) prying forces in angle

Figure 9–4. Illustration of variables in prying action calculations.

where

T = required strength, r_{ut} or r_{at}, per bolt, kips

$$b' = \left(b - \frac{d_b}{2}\right)$$

b = for a tee-type connecting element, the distance from bolt centerline to the face of the tee stem, in.; for an angle-type connecting element, the distance from bolt centerline to centerline of angle leg, in.

d_b = bolt diameter, in.

p = tributary length per pair of bolts for a tee or angle (perpendicular to the plane of the page in Figure 9–4), which should preferably not exceed the gage between the pair of bolts, g

F_u = specified minimum tensile strength of connecting element, ksi

When the resulting fitting thickness is reasonable, no further check of prying action is necessary. In this solution, the additional force in the bolt due to prying action, q, is essentially zero as long as the bolt force does not exceed T.

Alternatively, it is usually possible to determine a lesser required thickness by designing the connecting element and bolted joint for the actual effects of prying action with q greater than zero. To do so, a preliminary fitting thickness, t, can be selected based upon flexural yielding such that

LRFD	ASD
$T \le \dfrac{F_u t^2 p}{2.22b}$	$T \le \dfrac{F_u t^2 p}{3.33b}$

Table 15–1 can be used to select the preliminary fitting thickness. Subsequently, the thickness required to ensure an acceptable combination of fitting strength and stiffness and bolt strength, t_{min}, can be determined as

LRFD	ASD
$t_{min} = \sqrt{\dfrac{4.44Tb'}{pF_u\left(1+\delta\alpha'\right)}}$	$t_{min} = \sqrt{\dfrac{6.66Tb'}{pF_u\left(1+\delta\alpha'\right)}}$

where

$\delta = 1 - \dfrac{d'}{p}$ ratio of the net area at bolt line to gross area at face of the stem or leg of angle

$\alpha' = 1.0$ if $\beta \ge 1$

= the lesser of 1 and $\dfrac{1}{\delta}\left(\dfrac{\beta}{1-\beta}\right)$ if $\beta < 1$

d' = width of the hole along the length of the fitting, in.

$$\beta = \frac{1}{\rho}\left(\frac{B}{T} - 1\right)$$

$$\rho = \frac{b'}{a'}$$

$$a' = \left(a + \frac{d_b}{2}\right) \le \left(1.25b + \frac{d_b}{2}\right)$$

a = distance from the bolt centerline to the edge of the fitting, in.

B = available tension per bolt, ϕr_n or r_n/Ω, kips with $\phi = 0.75$ and $\Omega = 2.00$

If $t_{min} \le t$, the preliminary fitting thickness is satisfactory. Otherwise, a fitting with a thicker flange, or a change in geometry (i.e., b and p) is required.

Although it is not necessary to do so, if desired, the prying force per bolt, q, can be determined as

$$q = B\left[\delta\alpha\rho\left(\frac{t}{t_c}\right)^2\right]$$

$$\alpha = \frac{1}{\delta}\left[\frac{T}{B}\left(\frac{t_c}{t}\right)^2 - 1\right] \ge 0$$

LRFD	ASD
$t_c = \sqrt{\dfrac{4.44Bb'}{pF_u}}$	$t_c = \sqrt{\dfrac{6.66Bb'}{pF_u}}$

t_c = flange or angle thickness required to develop the available strength of the bolt, B, with no prying action, in.

The total force per bolt including the effects of prying action can then be determined as $T + q$.

Alternatively, when the fitting geometry is known, the available tensile strength per bolt, B, determined per AISC Specification Section J3.6, can be multiplied by Q to determine the available tensile strength including the effects of prying action

$$T_{avail} = BQ$$

where

Q = 1 if $\alpha' < 0$, which means that the fitting has sufficient strength and stiffness to develop the full bolt available tensile strength.

$= \left(\frac{t}{t_c}\right)^2 \left(1 + \delta\alpha'\right)$ if $0 \le \alpha' \le 1$, which means that the fitting has sufficient strength to develop the full bolt available tensile strength, but insufficient stiffness to prevent prying action.

$= \left(\dfrac{t}{t_c}\right)^2 \left(1+\delta\right)$ if $\alpha' > 1$, which means that the fitting has insufficient strength to develop the full bolt available tensile strength.

$\alpha' = \dfrac{1}{\delta\left(1+\rho\right)}\left[\left(\dfrac{t_c}{t}\right)^2 - 1\right]$ = value of α that either maximizes the bolt available tensile strength for a given thickness or minimizes the thickness required for a given bolt available tensile strength.

Rotational Ductility

Simple shear connections provide for the rotational ductility required by AISC Specification Section J1.2 as follows:

1. For double-angle, shear end-plate, single-angle, and tee shear connections, the geometry and thickness of the connecting elements attached to the support (angle legs, plate, or tee flange) are configured so that flexing of those connecting elements accommodates the simple-beam end rotation.
2. For unstiffened and stiffened seated connections, the geometry and thickness of the top or side stability angle is configured so that flexing of that connecting element accommodates the simple-beam end rotation.
3. For single-plate connections, the geometry and thickness of the plate are configured so that the plate will yield, bolt group will rotate, and/or or the bolt holes will elongate at failure prior to the failure of the welds or bolts.

For each of the simple-shear connections in Part 10, except tee shear connections, prescriptive guidance is provided to ensure adequate rotational ductility. Rotational ductility can be ensured for tee shear connections as follows. Note that this approach can also be used to demonstrate adequate rotational ductility in other simple shear connections that flex to accommodate the simple-beam end rotation, but with configurations that differ from those prescribed in Part 10.

When the connecting elements are welded to the support and bolted to the supported beam, weld size, w, with $F_{EXX} = 70$ ksi, must be such that

$$w_{min} = 0.0158\frac{F_y t_f^{\,2}}{b}\left(\frac{b^2}{L^2}+2\right)$$

but need not exceed $\frac{5}{8}t_s$, where

t_f = thickness of the tee flange, in.
t_s = thickness of the tee stem, in.
b = flexible width in connecting element as illustrated in Figure 9–5, in.
L = depth of connecting element as illustrated in Figure 9–5, in.

For a tee bolted to the support and bolted or welded to the supported beam, the minimum diameter for bolts through the tee flange for ductility must be such that

$$d_{b\,min} = 0.163 t_f \sqrt{\frac{F_y}{b}\left(\frac{b^2}{L^2} + 2\right)}$$

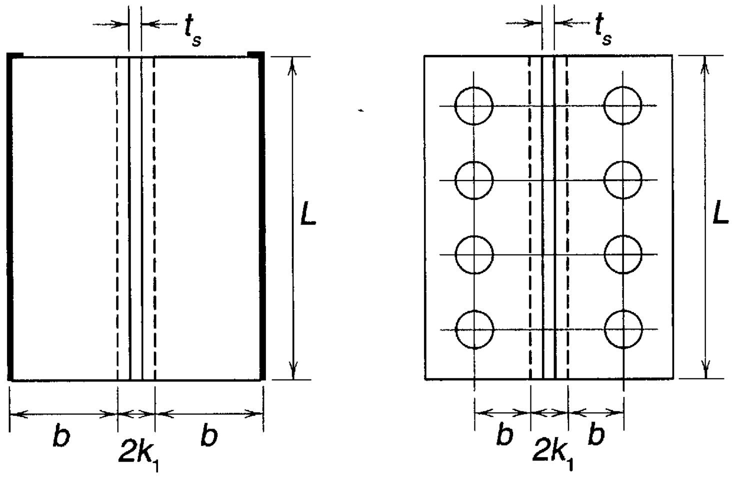
but need not exceed $0.69\sqrt{t_s}$. Additionally, to provide for rotational ductility when the tee stem is bolted to the supported beam, the maximum tee stem thickness should be such that

$$t_{s\,max} = \frac{d_b}{2} + {}^{1}/_{16}\text{ in.}$$

When the tee stem is welded to the supported beam, there is no perceived ductility problem for this weld.

Concentrated Forces

If the connecting element delivers a concentrated force to a member or other connecting element, see AISC Specification Section J10 or K1, as appropriate. See also AISC Design Guide No. 13 *Wide-Flange Column Stiffening at Moment Connections: Wind and Seismic Applications.*



Note: weld returns on top of tee per AISC Specification Section J2.2b

(a) Welded flange *(b) Bolted flange*

Figure 9–5. Illustration of variables in shear connection ductility checks.

Shims and Fillers

Shims are furnished to the erector for use in filling the spaces allowed for field clearance which might be present at connections such as simple shear connections, PR and FR moment connections, column base plates, and column splices. These shims, illustrated in Figure 9–6, may be either strip shims, with round punched holes, or finger shims, with slots cut through the edge. Whereas strip shims are less expensive to fabricate, finger shims may be laterally inserted and eliminate the need to remove erection bolts or pins already in place.

Finger shims, when inserted fully against the bolt shank, are acceptable for slip-critical connections and are not to be considered as an internal ply with the slotted hole determining the available strength of the connection. This is because less than 25 percent of the contact surface is lost and this is not enough to affect the performance of the joint.

A filler is furnished to occupy spaces which will be present because of dimensional separations between elements of a connection across which load transfer occurs. Examples where fillers might be used are beams framing off center on a column and raised beams.

For the effect of fillers and shims on available joint strength, see RCSC Specification Section 5.1.

Copes, Blocks, and Cuts

When structural members frame together, a minimum clearance of ½ in. should be provided, when possible. In cases where material removal is necessary to provide such a clearance, material may be removed by coping, blocking, or cutting as illustrated in Figure 9–7.

Material removal is costly and should be avoided when possible. In some cases, it may be possible to do so by setting the elevations of the tops of infill beams a sufficient distance below the tops of girders to clear the girder fillet radius. Alternatively, a connection such as that illustrated in Figure 9–8 could be used.

When material removal is necessary, coping is usually the most economical method to remove material. The recommended practices for coping are illustrated in Figure 9–9. The potential notch left by the first cut will occur in waste material and subsequently be removed by the second cut. All re-entrant corners must be shaped notch-free per AWS D1.1 to a radius. An approximate minimum radius to which this corner must be shaped is ½ in.

Copes, blocks, and cuts can significantly reduce the available strengths of members and may require web reinforcement; it may be more economical to use a heavier member than to provide such reinforcement. The available strength of the unreinforced coped member is determined from the limit states of flexural yielding, local buckling, and lateral torsional buckling, if applicable.

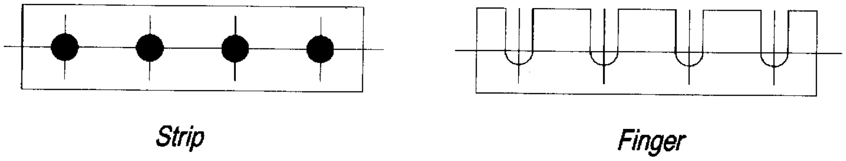
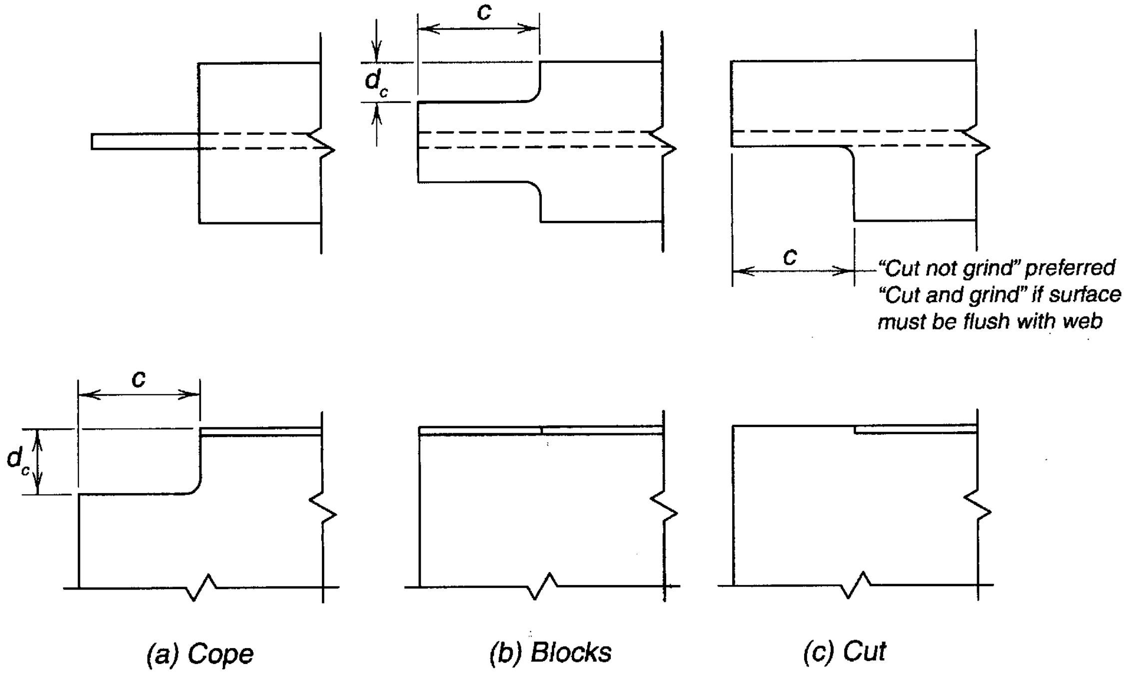
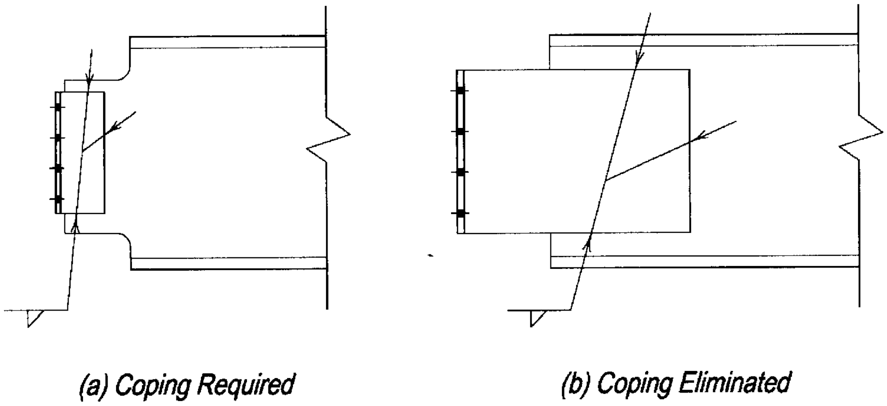


Figure 9–6. Shims.



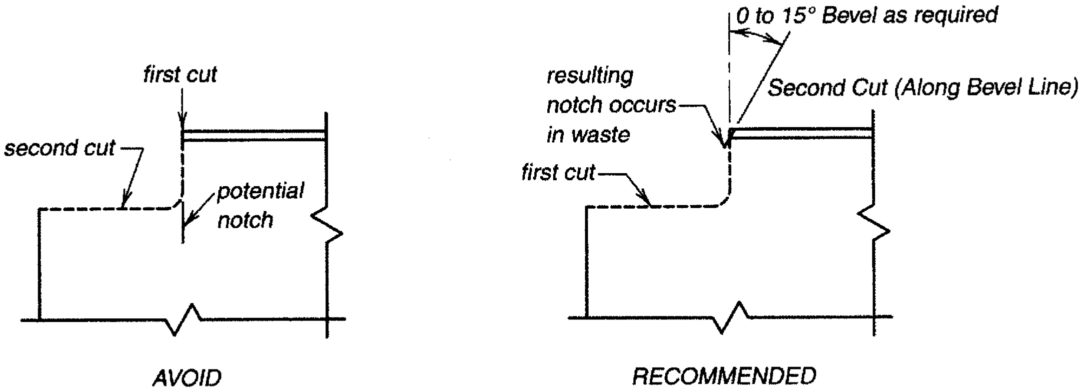
(a) Cope (b) Blocks (c) Cut

Figure 9–7. Copes, blocks, and cuts.



(a) Coping Required (b) Coping Eliminated

Figure 9–8. Minimizing coping requirements.



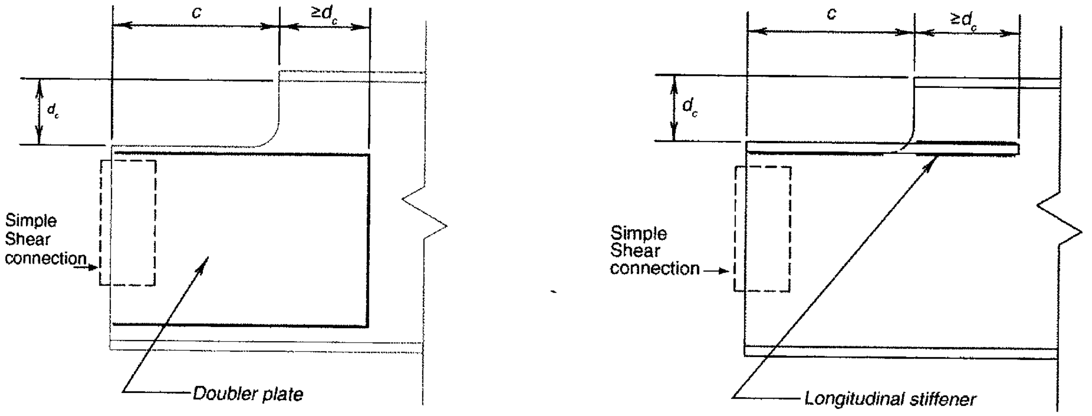
AVOID RECOMMENDED

Figure 9–9. Recommending coping practices.

Web Reinforcement of Coped Beams

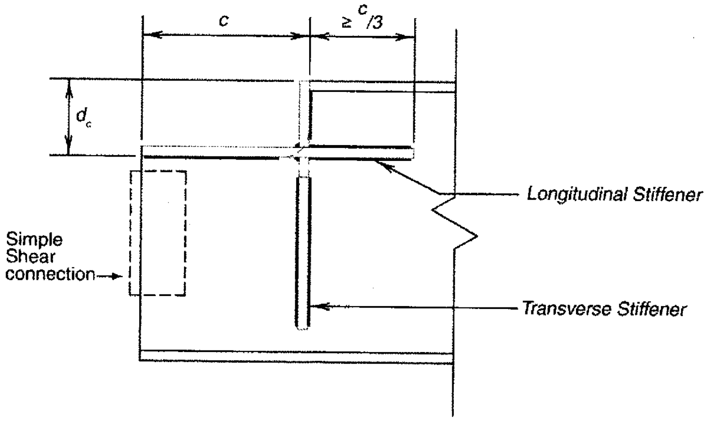
When the strength of a coped beam is inadequate, either a different beam can be selected to eliminate the need for reinforcement, or reinforcement can be provided to increase the strength. In spite of the increase in material cost, the former solution may be the most economical option due to the appreciable labor cost associated with adding stiffeners and/or doubler plates. When the latter solution is required, some typical reinforcing details are illustrated in Figure 9–10.

The doubler plate illustrated in Figure 9–10a and the longitudinal stiffener illustrated in Figure 9–10b are used with rolled sections where $h/t_w \leq 60$. When a doubler plate is used, the required doubler-plate thickness, $t_{d\,req}$, is determined by substituting the quantity $(t_w + t_{d\,req})$ for t_w in the available strength calculations for flexural yielding and local web buckling. To prevent local crippling of the beam web, the doubler plate must be extended at least a distance d_c (depth of cope) beyond the cope as illustrated in Figure 9–10a. When longitudinal stiffening is used, the stiffening elements must be proportioned to meet the width-thickness ratios specified in AISC Specification Table B4.1. The stiffened cross-section must then be checked for flexural yielding, but local web buckling need not be checked. To prevent local crippling of the beam web, the longitudinal stiffening must be extended a distance d_c beyond the cope as illustrated in Figure 9–10b.



(a) doubler plate

(b) Longitudinal stiffener



(c) Combination longitudinal and transverse stiffeners

Figure 9–10. Web reinforcement of coped beams.

The combination of longitudinal and transverse stiffeners shown in Figure 9–10c may be required for thin-webbed plate-girders, where $h/t_w > 60$. When longitudinal and transverse stiffening is used, the stiffening elements must be proportioned to meet the width-thickness ratios specified in AISC Specification Table B4.1. The stiffened cross-section must then be checked for flexural yielding, but local web buckling need not be checked. To prevent local crippling of the beam web, longitudinal stiffeners must be extended a distance $c/3$ beyond the cope, as illustrated in Figure 9–10c.

DESIGN TABLES

Table 9–1. Reduction in Area for Holes

Area reduction for standard, oversized, short-slotted, and long-slotted holes in material thicknesses from 3/16 in. to 1 in. are given in Table 9–1. For material thicknesses not listed, the tabular value for 1-in. thickness can be multiplied by the actual thickness.

Table 9–2. Elastic Section Modulus of Coped W-Shapes

Values are given for the gross and net elastic section moduli for coped W-shapes, as illustrated in the table header.

Tables 9–3. Block Shear Rupture

The terms in AISC Specification Equation J4-5 are tabulated in Tables 9–3a, 9–3b, and 9–3c. The indicated values are given per inch of material thickness. Note that when the stress distribution is non-uniform, the tension component from Table 9.3a must be reduced by factor of 0.5 to account for U_{bs}.

Table 9–4. Beam End Bearing Constants

At beam ends, the available strength for local web yielding, ϕR_n or R_n/Ω, is determined per AISC Specification Section J10.2. This can be simplified using the bearing length, N, and the constants R_1 through R_6, as outlined below.

$$R_1 = 2.5kF_{yw}t_w$$

$$R_2 = F_{yw}t_w$$

$$R_3 = 0.40t_w^2\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$R_4 = 0.40t_w^2\left(\frac{3}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$R_5 = 0.40t_w^2\left(1-0.2\left(\frac{t_w}{t_f}\right)^{1.5}\right)\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

$$R_6 = 0.40t_w^2\left(\frac{4}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\sqrt{\frac{EF_{yw}t_f}{t_w}}$$

Local Web Yielding at Beam Ends

At beam ends, the available strength for local web yielding, ϕR_n or R_n/Ω, is determined per AISC Specification Section J10.2 using Equations J10-2 or J10-3, which can be simplified using the constants R_1 and R_2 from Table 9–4 as follows:

When the compressive force to be resisted is applied at a distance, x, from the member end that is less than the depth of the member ($< d$),

LRFD	ASD
$\phi R_n = \phi R_1 + N(\phi R_2)$	$R_n/\Omega = R_1/\Omega + N(R_2/\Omega)$

When the compressive force to be resisted is applied at a distance, x, from the member end that is greater than or equal to the depth of the member ($\geq d$),

LRFD	ASD
$\phi R_n = 2(\phi R_1) + N(\phi R_2)$	$R_n/\Omega = 2(R_1/\Omega) + N(R_2/\Omega)$

Note that, as a minimum, the length of bearing, N, must be equal to k, per AISC Specification Section J10.2

Web Crippling at Beam Ends

At beam ends, the available strength for web crippling, R_n/Ω or ϕR_n, is determined per AISC Specification Section J10.3 using Equations J10-4, J10-5a, or J10-5b, which can be simplified using constants R_3, R_4, R_5, and R_6 from Table 9–4 as follows:

When the compressive force to be resisted is applied at a distance, x, from the member end that is less than one-half of the depth of the member ($< d/2$),

For $N/d \leq 0.2$:

LRFD	ASD
$\phi R_n = \phi R_3 + N(\phi R_4)$	$R_n/\Omega = R_3/\Omega + N(R_4/\Omega)$

For $N/d > 0.2$:

LRFD	ASD
$\phi R_n = \phi R_5 + N(\phi R_6)$	$R_n/\Omega = R_5/\Omega + N(R_6/\Omega)$

When the compressive force to be resisted is applied at a distance, x, from the member end that is greater than or equal to one-half of the depth of the member ($\geq d/2$),

LRFD	ASD
$\phi R_n = 2(\phi R_3) + N(\phi R_4)$	$R_n/\Omega = 2(R_3/\Omega) + N(R_4/\Omega)$

PART 9 REFERENCES

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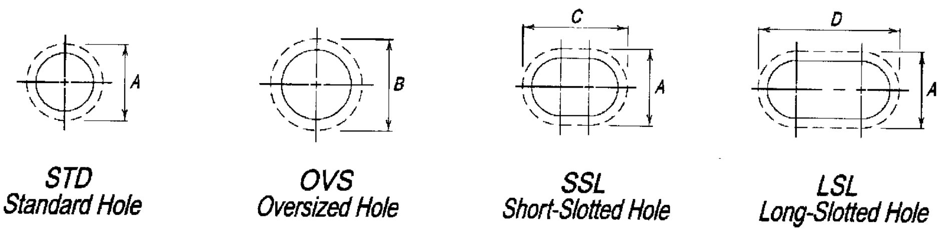
Table 9–1

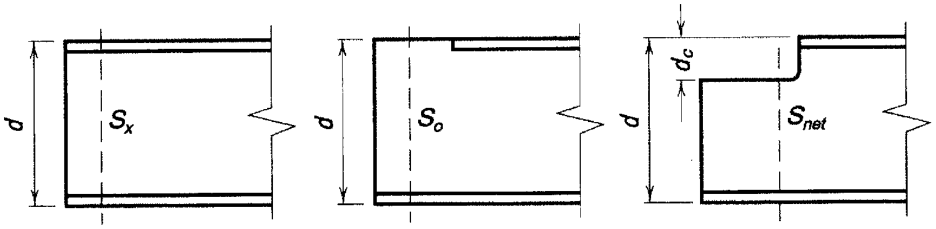
Reduction in Area for Holes, in.^{2}



Thickness t, in.	$A \times t$							$B \times t$						
	Bolt Diameter d_b, in.							Bolt Diameter d_b, in.						
	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
3/16	0.164	0.188	0.211	0.234	0.258	0.281	0.305	0.188	0.211	0.246	0.281	0.305	0.328	0.352
1/4	0.219	0.250	0.281	0.313	0.344	0.375	0.406	0.250	0.281	0.328	0.375	0.406	0.438	0.469
5/16	0.273	0.313	0.352	0.391	0.430	0.469	0.508	0.313	0.352	0.410	0.469	0.508	0.547	0.586
3/8	0.328	0.375	0.422	0.469	0.516	0.563	0.609	0.375	0.422	0.492	0.563	0.609	0.656	0.703
7/16	0.383	0.438	0.492	0.547	0.602	0.656	0.711	0.438	0.492	0.574	0.656	0.711	0.766	0.820
1/2	0.438	0.500	0.563	0.625	0.688	0.750	0.813	0.500	0.563	0.656	0.750	0.813	0.875	0.938
9/16	0.492	0.563	0.633	0.703	0.773	0.844	0.914	0.563	0.633	0.738	0.844	0.914	0.984	1.05
5/8	0.547	0.625	0.703	0.781	0.859	0.938	1.02	0.625	0.703	0.820	0.938	1.02	1.09	1.17
11/16	0.602	0.688	0.773	0.859	0.945	1.03	1.12	0.688	0.773	0.902	1.03	1.12	1.20	1.29
3/4	0.656	0.750	0.844	0.938	1.03	1.13	1.22	0.750	0.844	0.984	1.13	1.22	1.31	1.41
13/16	0.711	0.813	0.914	1.02	1.12	1.22	1.32	0.813	0.914	1.07	1.22	1.32	1.42	1.52
7/8	0.766	0.875	0.984	1.09	1.20	1.31	1.42	0.875	0.984	1.15	1.31	1.42	1.53	1.64
15/16	0.820	0.938	1.05	1.17	1.29	1.41	1.52	0.938	1.05	1.23	1.41	1.52	1.64	1.76
1	0.875	1.00	1.13	1.25	1.38	1.50	1.63	1.00	1.13	1.31	1.50	1.63	1.75	1.88

Thickness t, in.	$C \times t$							$D \times t$						
	Bolt Diameter d_b, in.							Bolt Diameter d_b, in.						
	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2	3/4	7/8	1	1 1/8	1 1/4	1 3/8	1 1/2
3/16	0.199	0.223	0.258	0.293	0.316	0.340	0.363	0.363	0.422	0.480	0.539	0.598	0.656	0.715
1/4	0.266	0.297	0.344	0.391	0.422	0.453	0.484	0.484	0.563	0.641	0.719	0.797	0.875	0.953
5/16	0.332	0.371	0.430	0.488	0.527	0.566	0.605	0.605	0.703	0.801	0.898	0.996	1.09	1.19
3/8	0.398	0.445	0.516	0.586	0.633	0.680	0.727	0.727	0.844	0.961	1.08	1.20	1.31	1.43
7/16	0.465	0.520	0.602	0.684	0.738	0.793	0.848	0.848	0.984	1.12	1.26	1.39	1.53	1.67
1/2	0.531	0.594	0.688	0.781	0.844	0.906	0.969	0.969	1.13	1.28	1.44	1.59	1.75	1.91
9/16	0.598	0.668	0.773	0.879	0.949	1.02	1.09	1.09	1.27	1.44	1.62	1.79	1.97	2.14
5/8	0.664	0.742	0.859	0.977	1.05	1.13	1.21	1.21	1.41	1.60	1.80	1.99	2.19	2.38
11/16	0.730	0.816	0.945	1.07	1.16	1.25	1.33	1.33	1.55	1.76	1.98	2.19	2.41	2.62
3/4	0.797	0.891	1.03	1.17	1.27	1.36	1.45	1.45	1.69	1.92	2.16	2.39	2.63	2.86
13/16	0.863	0.965	1.12	1.27	1.37	1.47	1.57	1.57	1.83	2.08	2.34	2.59	2.84	3.10
7/8	0.930	1.04	1.20	1.37	1.48	1.59	1.70	1.70	1.97	2.24	2.52	2.79	3.06	3.34
15/16	0.996	1.11	1.29	1.46	1.58	1.70	1.82	1.82	2.11	2.40	2.70	2.99	3.28	3.57
1	1.06	1.19	1.38	1.56	1.69	1.81	1.94	1.94	2.25	2.56	2.88	3.19	3.50	3.81

Table 9–2
Elastic Section Moduli for Coped W Shapes

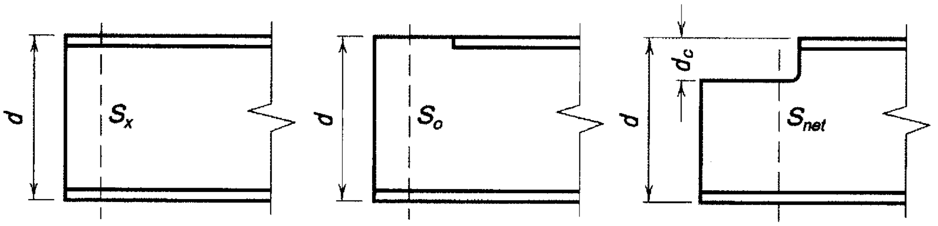


Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W44×335	44.0	1.77	1410	494	453	433	413	394	375	357	339	321	304
×290	43.6	1.58	1240	415	380	363	346	330	314	298	283	268	254
×262	43.3	1.42	1110	372	340	325	310	295	281	267	253	240	227
×230	42.9	1.22	971	330	301	288	274	261	249	236	224	212	200
W40×593	43.0	3.23	2340	810	—	—	671	639	607	575	545	515	486
×503	42.1	2.75	1980	671	—	582	554	527	500	473	448	423	398
×431	41.3	2.36	1690	567	—	491	467	444	421	398	376	355	334
×397	41.0	2.20	1560	512	—	444	422	400	379	359	339	319	300
×372	40.6	2.05	1460	480	—	415	394	374	354	335	316	298	280
×362	40.6	2.01	1420	463	—	400	380	361	342	323	305	287	270
×324	40.2	1.81	1280	408	371	352	335	317	300	284	268	252	237
×297	39.8	1.65	1170	374	339	323	306	290	275	259	245	230	216
×277	39.7	1.58	1100	335	304	289	274	260	246	232	219	206	193
×249	39.4	1.42	993	299	271	258	245	232	219	207	195	183	172
×215	39.0	1.22	859	256	231	220	208	197	186	176	166	156	146
×199	38.7	1.07	770	247	224	213	202	191	180	170	160	150	141
W40×392	41.6	2.52	1440	579	—	503	478	454	431	408	386	364	343
×331	40.8	2.13	1210	483	—	419	398	378	358	339	320	302	284
×327	40.8	2.13	1200	470	—	407	387	367	348	329	311	293	276
×294	40.4	1.93	1080	417	379	360	342	325	308	291	275	259	243
×278	40.2	1.81	1020	397	361	344	326	310	293	277	262	246	232
×264	40.0	1.73	971	371	337	321	305	289	274	259	244	230	216
×235	39.7	1.58	875	320	291	276	262	249	235	222	210	197	185
×211	39.4	1.42	786	286	259	246	234	221	209	198	186	175	165
×183	39.0	1.20	675	243	221	210	199	188	178	168	158	149	140
×167	38.6	1.03	600	234	212	201	191	181	171	161	152	143	134
×149	38.2	0.830	513	217	196	186	177	167	158	149	140	132	123

—Indicates that cope depth is less than flange thickness.

Table 9–2 (continued)

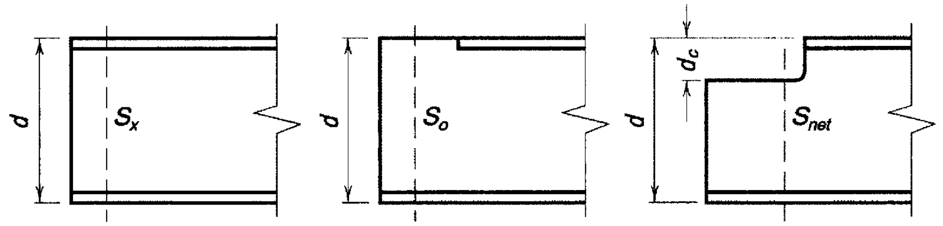
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W36×800	42.6	4.29	3040	1040	—	—	—	818	776	735	695	656	618
×652	41.1	3.54	2460	816	—	—	669	635	601	568	536	505	475
×529	39.8	2.91	1990	636	—	547	519	491	464	438	413	388	364
×487	39.3	2.68	1830	581	—	499	473	448	423	399	375	352	330
×441	38.9	2.44	1650	518	—	444	420	398	375	354	332	312	292
×395	38.4	2.20	1490	457	—	391	370	350	330	311	292	274	256
×361	38.0	2.01	1350	412	—	352	333	315	297	279	262	246	230
×330	37.7	1.85	1240	371	335	317	300	283	267	251	235	220	206
×302	37.3	1.68	1130	338	305	289	273	258	243	228	214	200	187
×282	37.1	1.57	1050	314	283	268	253	239	225	211	198	185	173
×262	36.9	1.44	972	294	264	250	236	223	210	197	185	172	161
×247	36.7	1.35	913	277	249	236	223	210	198	185	174	162	151
×231	36.5	1.26	854	260	234	222	209	197	186	174	163	152	142
W36×256	37.4	1.73	895	329	297	281	266	251	237	223	209	196	183
×232	37.1	1.57	809	295	266	251	238	224	211	199	186	174	163
×210	36.7	1.36	719	272	245	232	219	207	195	183	172	161	150
×194	36.5	1.26	664	249	224	212	201	189	178	167	157	146	137
×182	36.3	1.18	623	234	211	199	188	178	167	157	147	137	128
×170	36.2	1.10	581	218	196	185	175	165	155	146	137	128	119
×160	36.0	1.02	542	206	185	175	165	156	147	138	129	120	112
×150	35.9	0.940	504	195	176	166	157	148	139	130	122	114	106
×135	35.6	0.790	439	181	163	154	145	137	129	121	113	105	98.1
W33×387	36.0	2.28	1350	413	—	349	329	310	291	272	254	237	220
×354	35.6	2.09	1240	373	—	315	297	279	262	245	229	213	198
×318	35.2	1.89	1110	330	295	278	262	246	230	216	201	187	173
×291	34.8	1.73	1020	300	268	253	238	223	209	195	182	169	157
×263	34.5	1.57	919	268	239	226	212	199	186	174	162	151	139
×241	34.2	1.40	831	250	223	210	197	185	173	162	150	140	129
×221	33.9	1.28	759	230	205	193	181	170	159	148	138	128	118
×201	33.7	1.15	686	209	186	175	165	154	144	135	125	116	107

—Indicates that cope depth is less than flange thickness.

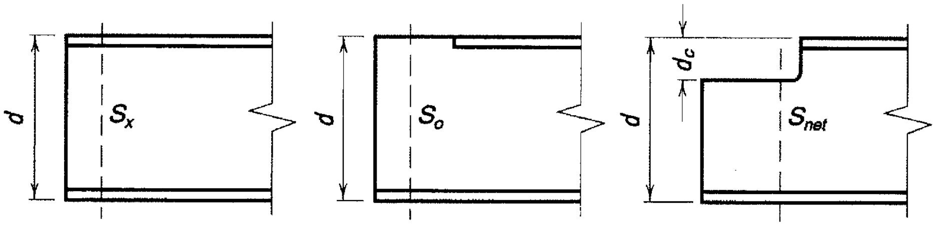
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d	t_f	S_x	S_o	S_{net}, in.^{3}								
	in.	in.	in.^{3}	in.^{3}	d_c, in.								
					2	3	4	5	6	7	8	9	10
W33×169	33.8	1.22	549	191	170	161	151	141	132	124	115	107	98.6
×152	33.5	1.06	487	176	157	148	139	130	122	114	106	97.9	90.5
×141	33.3	0.960	448	165	147	139	130	122	114	106	98.8	91.6	84.6
×130	33.1	0.855	406	155	138	130	122	114	107	99.6	92.5	85.7	79.2
×118	32.9	0.740	359	143	128	120	113	106	98.6	91.9	85.4	79.1	73.0
W30×391	33.2	2.44	1250	378	—	315	295	276	257	239	222	205	188
×357	32.8	2.24	1140	339	—	282	264	246	230	213	197	182	167
×326	32.4	2.05	1040	305	—	254	237	221	206	191	177	163	150
×292	32.0	1.85	930	269	238	223	208	194	180	167	155	142	130
×261	31.6	1.65	829	240	212	198	185	172	160	148	137	126	115
×235	31.3	1.50	748	211	186	174	163	152	141	130	120	110	101
×211	30.9	1.32	665	192	170	159	148	138	128	118	109	99.8	91.2
×191	30.7	1.19	600	174	153	143	133	124	115	106	97.7	89.6	81.8
×173	30.4	1.07	541	158	139	130	121	112	104	96.1	88.4	81.0	73.9
×148	30.7	1.18	436	152	134	125	117	109	101	93.3	86.0	78.9	72.1
×132	30.3	1.00	380	139	123	115	107	99.3	92.1	85.1	78.3	71.8	65.5
×124	30.2	0.930	355	131	115	108	100	93.4	86.5	79.9	73.6	67.4	61.5
×116	30.0	0.850	329	124	109	102	95.3	88.6	82.1	75.8	69.7	63.9	58.2
×108	29.8	0.760	299	118	103	96.5	89.9	83.6	77.4	71.4	65.7	60.1	54.8
×99	29.7	0.670	269	110	96.4	90.0	83.9	77.9	72.1	66.5	61.1	56.0	51.0
×90	29.5	0.610	245	98.7	86.7	80.9	75.4	70.0	64.8	59.7	54.9	50.2	45.7
W27×539	32.5	3.54	1570	509	—	—	394	367	341	316	292	269	247
×368	30.4	2.48	1060	321	—	262	244	226	209	193	177	162	147
×336	30.0	2.28	972	287	—	234	218	202	186	172	157	143	130
×307	29.6	2.09	887	259	—	211	196	181	167	154	141	128	116
×281	29.3	1.93	814	233	203	189	176	162	150	137	126	114	104
×258	29.0	1.77	745	212	185	172	159	147	136	124	114	103	93.3
×235	28.7	1.61	677	193	168	156	145	134	123	113	103	93.2	84.2
×217	28.4	1.50	627	174	152	141	130	120	111	101	92.3	83.7	75.5
×194	28.1	1.34	559	155	134	125	115	106	97.6	89.3	81.3	73.6	66.3
×178	27.8	1.19	505	145	126	117	108	99.7	91.5	83.6	76.1	68.8	61.9
×161	27.6	1.08	458	131	113	105	97.2	89.5	82.0	74.9	68.1	61.5	55.3
×146	27.4	0.975	414	118	102	95.0	87.7	80.7	74.0	67.5	61.3	55.3	49.7

—Indicates that cope depth is less than flange thickness.

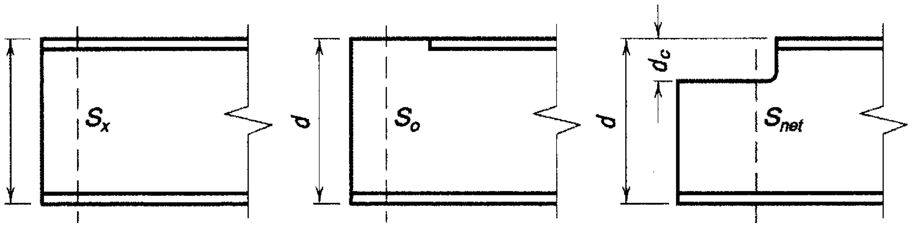
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W27×129	27.6	1.10	345	117	101	94.0	86.9	80.1	73.5	67.2	61.1	55.3	49.7
×114	27.3	0.930	299	106	91.6	84.9	78.4	72.2	66.2	60.5	54.9	49.6	44.6
×102	27.1	0.830	267	94.2	81.6	75.6	69.8	64.2	58.9	53.7	48.8	44.0	39.5
×94	26.9	0.745	243	88.0	76.2	70.6	65.1	59.9	54.9	50.1	45.4	41.0	36.8
×84	26.7	0.640	213	80.5	69.7	64.5	59.5	54.7	50.1	45.7	41.4	37.4	33.5
W24×370	28.0	2.72	957	295	—	237	219	201	184	168	153	138	124
×335	27.5	2.48	864	261	—	209	193	177	162	147	133	120	108
×306	27.1	2.28	789	234	—	186	172	157	144	131	118	106	94.9
×279	26.7	2.09	718	210	—	167	154	141	128	116	105	94.3	84.0
×250	26.3	1.89	644	184	158	146	134	123	112	101	91.2	81.7	72.6
×229	26.0	1.73	588	167	143	132	121	111	101	91.0	81.8	73.1	64.9
×207	25.7	1.57	531	149	127	117	107	98.0	89.0	80.4	72.2	64.4	57.0
×192	25.5	1.46	491	136	117	107	98.2	89.5	81.2	73.3	65.8	58.6	51.8
×176	25.2	1.34	450	124	106	97.6	89.4	81.4	73.8	66.5	59.6	53.0	46.8
×162	25.0	1.22	414	115	98.0	90.0	82.3	74.9	67.9	61.1	54.7	48.6	42.8
×146	24.7	1.09	371	104	88.5	81.2	74.2	67.5	61.1	54.9	49.1	43.6	38.3
×131	24.5	0.960	329	94.4	80.3	73.7	67.3	61.1	55.3	49.7	44.3	39.3	34.5
×117	24.3	0.850	291	84.4	71.7	65.7	60.0	54.5	49.2	44.2	39.4	34.8	30.5
×104	24.1	0.750	258	75.4	64.1	58.7	53.5	48.6	43.8	39.3	35.0	30.9	27.1
W24×103	24.5	0.980	245	82.9	70.7	64.9	59.3	53.9	48.8	43.9	39.2	34.8	30.6
×94	24.3	0.875	222	76.2	64.9	59.5	54.3	49.4	44.6	40.1	35.8	31.7	27.9
×84	24.1	0.770	196	68.3	58.0	53.2	48.6	44.1	39.8	35.8	31.9	28.2	24.8
×76	23.9	0.680	176	62.6	53.2	48.7	44.5	40.4	36.4	32.7	29.1	25.8	22.6
×68	23.7	0.585	154	57.5	48.8	44.7	40.8	37.0	33.4	29.9	26.6	23.5	20.6
×62	23.7	0.590	131	56.9	48.3	44.3	40.4	36.7	33.1	29.7	26.5	23.4	20.5
×55	23.6	0.505	114	51.1	43.4	39.7	36.2	32.9	29.7	26.6	23.7	20.9	18.3
W21×201	23.0	1.63	461	125	105	95.2	86.2	77.6	69.4	61.6	54.2	47.3	40.8
×182	22.7	1.48	417	111	93.3	84.8	76.6	68.8	61.4	54.4	47.8	41.6	35.8
×166	22.5	1.36	380	99.3	83.0	75.3	68.0	61.0	54.4	48.1	42.2	36.6	31.4
×147	22.1	1.15	329	91.2	76.1	68.9	62.1	55.7	49.5	43.7	38.2	33.1	28.2
×132	21.8	1.04	295	81.0	67.5	61.1	55.0	49.2	43.7	38.5	33.6	29.0	24.7
×122	21.7	0.960	273	74.1	61.6	55.7	50.2	44.8	39.8	35.0	30.5	26.3	22.4
×111	21.5	0.875	249	67.1	55.7	50.4	45.3	40.4	35.9	31.5	27.4	23.6	20.1
×101	21.4	0.800	227	60.4	50.1	45.3	40.7	36.3	32.1	28.2	24.5	21.1	17.9

—Indicates that cope depth is less than flange thickness.

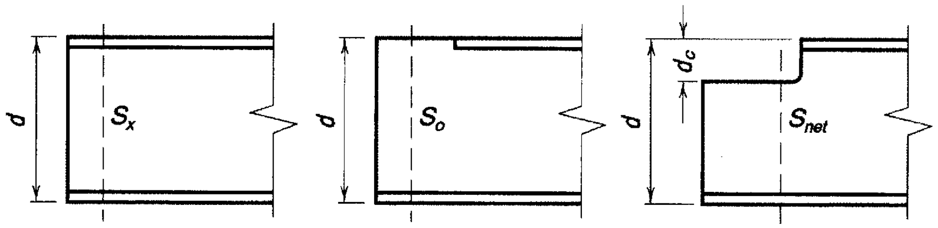
Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W21×93	21.6	0.930	192	67.2	56.0	50.7	45.7	40.9	36.3	32.0	27.9	24.1	20.5
×83	21.4	0.835	171	59.0	49.1	44.4	40.0	35.7	31.7	27.9	24.3	20.9	17.8
×73	21.2	0.740	151	51.5	42.7	38.7	34.8	31.0	27.5	24.2	21.0	18.1	15.3
×68	21.1	0.685	140	48.1	39.9	36.1	32.4	29.0	25.6	22.5	19.6	16.8	14.2
×62	21.0	0.615	127	44.1	36.5	33.0	29.7	26.5	23.4	20.5	17.8	15.3	12.9
×55	20.8	0.522	110	40.1	33.2	30.0	26.9	24.0	21.2	18.6	16.1	13.8	11.7
×48	20.6	0.430	93.0	36.2	30.0	27.0	24.2	21.6	19.1	16.7	14.5	12.4	10.4
W21×57	21.1	0.650	111	43.4	36.1	32.6	29.3	26.2	23.2	20.4	17.7	15.2	12.9
×50	20.8	0.535	94.5	39.2	32.5	29.4	26.4	23.6	20.8	18.3	15.9	13.6	11.5
×44	20.7	0.450	81.6	35.2	29.1	26.3	23.6	21.0	18.6	16.3	14.1	12.1	10.2
W18×311	22.3	2.74	624	186	—	140	126	113	100	88.2	77.0	66.5	56.8
×283	21.9	2.50	565	166	—	124	111	99.3	87.8	77.1	67.0	57.6	48.9
×258	21.5	2.30	514	148	—	110	98.3	87.4	77.2	67.5	58.5	50.0	42.3
×234	21.1	2.11	466	130	—	96.1	85.9	76.2	67.1	58.5	50.4	43.0	36.1
×211	20.7	1.91	419	115	94.5	84.8	75.6	66.9	58.7	51.0	43.8	37.1	31.0
×192	20.4	1.75	380	102	83.4	74.7	66.5	58.7	51.4	44.5	38.1	32.1	26.7
×175	20.0	1.59	344	92.1	75.1	67.2	59.7	52.6	45.9	39.6	33.8	28.4	23.5
×158	19.7	1.44	310	81.7	66.4	59.3	52.6	46.2	40.2	34.6	29.4	24.6	
×143	19.5	1.32	282	72.5	58.8	52.4	46.4	40.7	35.4	30.4	25.7	21.5	
×130	19.3	1.20	256	65.2	52.8	47.0	41.5	36.4	31.5	27.0	22.8	19.0	
×119	19.0	1.06	231	61.7	49.8	44.3	39.1	34.2	29.5	25.2	21.2	17.6	
×106	18.7	0.940	204	54.4	43.8	38.9	34.3	29.9	25.8	22.0	18.5	15.2	
×97	18.6	0.870	188	48.9	39.3	34.9	30.7	26.8	23.1	19.6	16.4	13.5	
×86	18.4	0.770	166	43.1	34.6	30.6	26.9	23.4	20.2	17.1	14.3	11.7	
×76	18.2	0.680	146	37.6	30.1	26.7	23.4	20.3	17.5	14.8	12.3	10.1	
W18×71	18.5	0.810	127	42.4	34.1	30.3	26.7	23.3	20.1	17.1	14.3	11.8	
×65	18.4	0.750	117	38.3	30.8	27.3	24.0	20.9	18.0	15.3	12.8	10.5	
×60	18.2	0.695	108	35.0	28.1	24.9	21.9	19.1	16.4	13.9	11.6	9.53	
×55	18.1	0.630	98.3	32.4	26.0	23.0	20.2	17.6	15.1	12.8	10.7	8.72	
×50	18.0	0.570	88.9	29.1	23.4	20.7	18.2	15.8	13.5	11.5	9.54		
W18×46	18.1	0.605	78.8	28.9	23.2	20.6	18.1	15.7	13.5	11.5	9.56	7.81	
×40	17.9	0.525	68.4	24.9	20.0	17.7	15.5	13.5	11.6	9.80	8.16		
×35	17.7	0.425	57.6	22.7	18.2	16.1	14.1	12.3	10.5	8.88	7.37		

—Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds $d/2$.

Table 9-2 (continued)
Elastic Section Moduli for Coped W Shapes

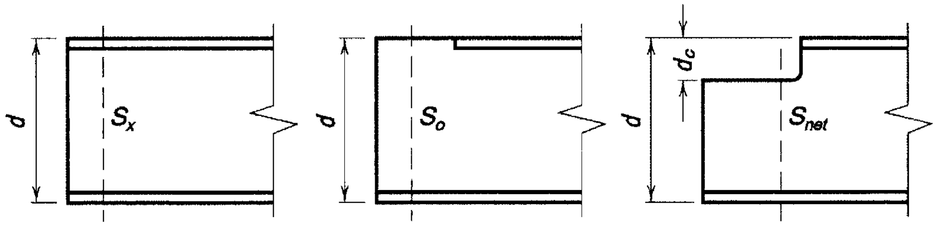


Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W16×100	17.0	0.985	175	44.4	34.9	30.5	26.4	22.6	19.0	15.7	12.8		
×89	16.8	0.875	155	39.0	30.6	26.7	23.1	19.7	16.5	13.6	11.0		
×77	16.5	0.760	134	33.1	25.9	22.6	19.4	16.5	13.8	11.4	9.13		
×67	16.3	0.665	117	28.3	22.1	19.2	16.5	14.0	11.7	9.58	7.66		
W16×57	16.4	0.715	92.2	29.4	23.0	20.1	17.3	14.8	12.4	10.2	8.17		
×50	16.3	0.630	81.0	25.6	20.0	17.4	15.0	12.7	10.7	8.74	6.99		
×45	16.1	0.565	72.7	22.9	17.9	15.5	13.4	11.3	9.47	7.75	6.19		
×40	16.0	0.505	64.7	20.1	15.6	13.6	11.7	9.89	8.24	6.73	5.35		
×36	15.9	0.430	56.5	18.8	14.6	12.7	10.9	9.21	7.67	6.25			
W16×31	15.9	0.440	47.2	17.1	13.3	11.6	9.96	8.44	7.03	5.73			
×26	15.7	0.345	38.4	14.9	11.6	10.1	8.64	7.31	6.08	4.95			
W14×730	22.4	4.91	1280	365	—	—	—	220	195	172	151	132	114
×665	21.6	4.52	1150	317	—	—	—	187	165	144	126	109	93.3
×605	20.9	4.16	1040	275	—	—	—	158	139	121	105	89.6	76.2
×550	20.2	3.82	931	238	—	—	153	134	117	101	86.9	73.8	62.1
×500	19.6	3.50	838	208	—	—	131	115	99.4	85.3	72.5	60.9	
×455	19.0	3.21	756	182	—	—	113	98.2	84.6	72.1	60.7	50.6	
×426	18.7	3.04	706	164	—	—	101	87.6	75.2	63.8	53.4	44.2	
×398	18.3	2.85	656	150	—	104	91.1	78.7	67.2	56.7	47.2	38.7	
×370	17.9	2.66	607	135	—	93.7	81.4	70.1	59.6	50.0	41.3		
×342	17.5	2.47	558	122	—	83.4	72.3	61.9	52.3	43.6	35.8		
×311	17.1	2.26	506	107	—	72.7	62.7	53.5	44.9	37.2	30.2		
×283	16.7	2.07	459	94.4	—	63.6	54.6	46.3	38.7	31.8	25.6		
×257	16.4	1.89	415	83.1	64.1	55.5	47.4	40.0	33.3	27.1	21.6		
×233	16.0	1.72	375	73.2	56.1	48.4	41.3	34.6	28.6	23.2	18.3		
×211	15.7	1.56	338	64.9	49.5	42.6	36.1	30.2	24.8	19.9			
×193	15.5	1.44	310	57.6	43.8	37.5	31.7	26.4	21.6	17.3			
×176	15.2	1.31	281	52.2	39.5	33.8	28.5	23.6	19.2	15.2			
×159	15.0	1.19	254	45.7	34.5	29.4	24.7	20.4	16.5	13.0			
×145	14.8	1.09	232	40.9	30.7	26.1	21.9	18.0	14.5	11.4			

—Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds $d/2$.

Table 9–2 (continued)

Elastic Section Moduli for Coped W Shapes

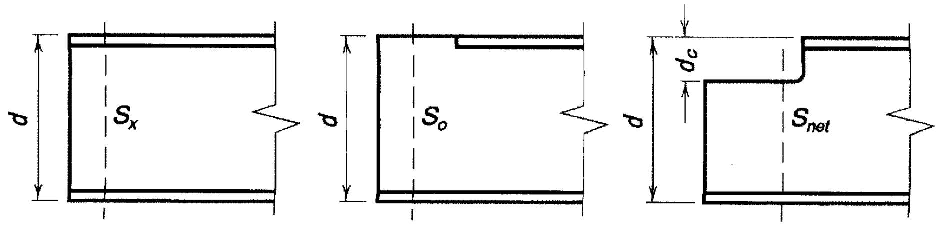


Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W14×132	14.7	1.03	209	38.1	28.6	24.3	20.3	16.7	13.4	10.5			
×120	14.5	0.940	190	34.2	25.5	21.7	18.1	14.8	11.8	9.20			
×109	14.3	0.860	173	30.0	22.3	18.9	15.7	12.8	10.2	7.91			
×99	14.2	0.780	157	27.2	20.2	17.0	14.2	11.5	9.15	7.04			
×90	14.0	0.710	143	24.3	18.0	15.2	12.6	10.2	8.07	6.18			
W14×82	14.3	0.855	123	28.0	20.9	17.7	14.8	12.1	9.64	7.46			
×74	14.2	0.785	112	24.4	18.2	15.4	12.8	10.4	8.31	6.40			
×68	14.0	0.720	103	22.2	16.5	13.9	11.6	9.41	7.46	5.72			
×61	13.9	0.645	92.1	19.7	14.6	12.3	10.2	8.28	6.54				
W14×53	13.9	0.660	77.8	19.1	14.2	12.0	9.93	8.07	6.39				
×48	13.8	0.595	70.2	17.3	12.8	10.8	8.93	7.23	5.71				
×43	13.7	0.530	62.6	15.3	11.3	9.49	7.84	6.34	4.99				
W14×38	14.1	0.515	54.6	16.0	12.0	10.2	8.48	6.94	5.54	4.28			
×34	14.0	0.455	48.6	14.4	10.8	9.14	7.62	6.22	4.95				
×30	13.8	0.385	42.0	13.2	9.88	8.37	6.96	5.68	4.51				
W14×26	13.9	0.420	35.3	12.3	9.20	7.80	6.50	5.31	4.23				
×22	13.7	0.335	29.0	10.7	7.97	6.75	5.62	4.58	3.64				
W12×336	16.8	2.96	483	123	—	83.1	71.4	60.6	50.8	41.9	34.1		
×305	16.3	2.71	435	108	—	71.4	61.0	51.4	42.7	34.9	28.0		
×279	15.9	2.47	393	96.1	—	63.1	53.5	44.8	36.9	29.8			
×252	15.4	2.25	353	83.7	—	54.2	45.7	38.0	31.0	24.8			
×230	15.1	2.07	321	74.2	—	47.5	39.9	32.9	26.7	21.1			
×210	14.7	1.90	292	65.6	49.0	41.6	34.7	28.5	22.9	17.9			
×190	14.4	1.74	263	57.0	42.3	35.7	29.7	24.2	19.3	14.9			
×170	14.0	1.56	235	49.6	36.5	30.7	25.3	20.5	16.2	12.4			
×152	13.7	1.40	209	43.3	31.6	26.5	21.7	17.5	13.7				
×136	13.4	1.25	186	37.9	27.5	22.9	18.7	14.9	11.6				
×120	13.1	1.11	163	32.8	23.7	19.7	16.0	12.6	9.70				
×106	12.9	0.990	145	27.6	19.8	16.3	13.2	10.4	7.91				
×96	12.7	0.900	131	24.3	17.4	14.3	11.5	9.03	6.83				
×87	12.5	0.810	118	22.2	15.8	13.0	10.4	8.11	6.09				
×79	12.4	0.735	107	19.9	14.1	11.5	9.23	7.16	5.35				
×72	12.3	0.670	97.4	17.9	12.6	10.3	8.24	6.37	4.73				
×65	12.1	0.605	87.9	16.0	11.2	9.16	7.28	5.61	4.14				

—Indicates that cope depth is less than flange thickness.
Note: Values are omitted when cope depth exceeds $d/2$.

Table 9–2 (continued)

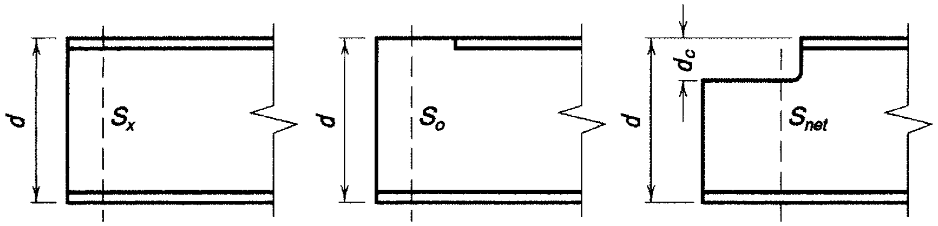
Elastic Section Moduli for Coped W Shapes



Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W12×58	12.2	0.640	78.0	14.8	10.4	8.52	6.79	5.24	3.88				
×53	12.1	0.575	70.6	13.9	9.75	7.94	6.31	4.85	3.58				
W12×50	12.2	0.640	64.2	14.8	10.4	8.54	6.82	5.27	3.91				
×45	12.1	0.575	57.7	13.1	9.27	7.56	6.02	4.63	3.42				
×40	11.9	0.515	51.5	11.4	8.03	6.54	5.19	3.98					
W12×35	12.5	0.520	45.6	12.3	8.85	7.30	5.89	4.61	3.48				
×30	12.3	0.440	38.6	10.5	7.47	6.15	4.94	3.86	2.90				
×26	12.2	0.380	33.4	9.08	6.47	5.32	4.27	3.32	2.48				
W12×22	12.3	0.425	25.4	9.60	6.89	5.69	4.59	3.59	2.71				
×19	12.2	0.350	21.3	8.39	6.01	4.95	3.98	3.11	2.33				
×16	12.0	0.265	17.1	7.43	5.30	4.36	3.50	2.72					
×14	11.9	0.225	14.9	6.61	4.71	3.86	3.10	2.41					
W10×112	11.4	1.25	126	25.7	17.5	13.9	10.8	8.02					
×100	11.1	1.12	112	22.3	15.0	11.9	9.12	6.72					
×88	10.8	0.990	98.5	19.1	12.8	10.0	7.62	5.54					
×77	10.6	0.870	85.9	16.2	10.7	8.35	6.29	4.52					
×68	10.4	0.770	75.7	13.9	9.13	7.10	5.30	3.77					
×60	10.2	0.680	66.7	12.1	7.88	6.09	4.52	3.18					
×54	10.1	0.615	60.0	10.5	6.78	5.22	3.85	2.69					
×49	10.0	0.560	54.6	9.49	6.13	4.71	3.46	2.40					
W10×45	10.1	0.620	49.1	9.75	6.33	4.88	3.61	2.52					
×39	9.92	0.530	42.1	8.49	5.48	4.20	3.08						
×33	9.73	0.435	35.0	7.49	4.80	3.67	2.67						
W10×30	10.5	0.510	32.4	8.64	5.75	4.51	3.41	2.45					
×26	10.3	0.440	27.9	7.33	4.86	3.80	2.85	2.04					
×22	10.2	0.360	23.2	6.51	4.29	3.34	2.50	1.77					
W10×19	10.2	0.395	18.8	6.52	4.33	3.39	2.55	1.82					
×17	10.1	0.330	16.2	6.01	3.98	3.10	2.33	1.65					
×15	10.0	0.270	13.8	5.53	3.65	2.84	2.12	1.50					
×12	9.87	0.210	10.9	4.43	2.91	2.26	1.68						

Note: Values are omitted when cope depth exceeds $d/2$.

Table 9–2 (continued)
Elastic Section Moduli for Coped W Shapes



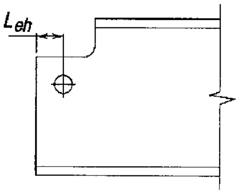
Shape	d in.	t_f in.	S_x in.^{3}	S_o in.^{3}	S_{net}, in.^{3}								
					d_c, in.								
					2	3	4	5	6	7	8	9	10
W8×67	9.00	0.935	60.4	12.2	7.42	5.44	3.77						
×58	8.75	0.810	52.0	10.4	6.24	4.52	3.08						
×48	8.50	0.685	43.2	7.89	4.63	3.32	2.21						
×40	8.25	0.560	35.5	6.71	3.89	2.74	1.80						
×35	8.12	0.495	31.2	5.66	3.24	2.28	1.47						
×31	8.00	0.435	27.5	5.06	2.88	2.01	1.28						
W8×28	8.06	0.465	24.3	5.04	2.89	2.02	1.30						
×24	7.93	0.400	20.9	4.23	2.40	1.67							
W8×21	8.28	0.400	18.2	4.55	2.67	1.91	1.26						
×18	8.14	0.330	15.2	4.02	2.35	1.66	1.09						
W8×15	8.11	0.315	11.8	4.03	2.36	1.68	1.10						
×13	7.99	0.255	9.91	3.61	2.10	1.49							
×10	7.89	0.205	7.81	2.65	1.54	1.08							

Note: Values are omitted when cope depth exceeds $d/2$.

U_{bs} = 1.0

Table 9–3a
Block Shear Tension Rupture Component

per inch of thickness, kips/in.

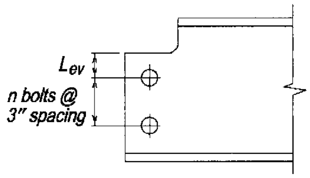


F_u	58 ksi					
L_{eh}, in.	Bolt diameter, d_b, in.					
	3/4		7/8		1	
	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	16.3	24.5	14.5	21.8	12.7	19.0
1⅛	19.9	29.9	18.1	27.2	16.3	24.5
1¼	23.6	35.3	21.8	32.6	19.9	29.9
1⅜	27.2	40.8	25.4	38.1	23.6	35.3
1½	30.8	46.2	29.0	43.5	27.2	40.8
1⅝	34.4	51.7	32.6	48.9	30.8	46.2
1¾	38.1	57.1	36.3	54.4	34.4	51.7
1⅞	41.7	62.5	39.9	59.8	38.1	57.1
2	45.3	68.0	43.5	65.3	41.7	62.5
2¼	52.6	78.8	50.7	76.1	48.9	73.4
2½	59.8	89.7	58.0	87.0	56.2	84.3
2¾	67.1	101	65.3	97.9	63.4	95.2
3	74.3	111	72.5	109	70.7	106

F_u	65 ksi					
L_{eh}, in.	Bolt diameter, d_b, in.					
	3/4		7/8		1	
	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$	$\frac{F_uA_{nt}}{t\Omega}$	$\frac{\phi F_uA_{nt}}{t}$
	ASD	LRFD	ASD	LRFD	ASD	LRFD
1	18.3	27.4	16.3	24.4	14.2	21.3
1⅛	22.3	33.5	20.3	30.5	18.3	27.4
1¼	26.4	39.6	24.4	36.6	22.3	33.5
1⅜	30.5	45.7	28.4	42.7	26.4	39.6
1½	34.5	51.8	32.5	48.8	30.5	45.7
1⅝	38.6	57.9	36.6	54.8	34.5	51.8
1¾	42.7	64.0	40.6	60.9	38.6	57.9
1⅞	46.7	70.1	44.7	67.0	42.7	64.0
2	50.8	76.2	48.8	73.1	46.7	70.1
2¼	58.9	88.4	56.9	85.3	54.8	82.3
2½	67.0	101	65.0	97.5	63.0	94.5
2¾	75.2	113	73.1	110	71.1	107
3	83.3	125	81.3	122	79.2	119

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

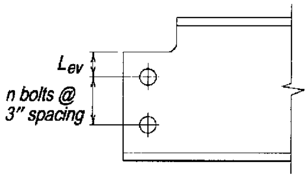
Table 9–3b
Block Shear
Shear Yielding Component
per inch of thickness, kips/in.



L_{ev}, in.	n	F_y, ksi				n	F_y, ksi			
		36		50			36		50	
		$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$	$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$		$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$	$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$
		ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD
1¼	12	370	555	514	771	9	273	409	379	568
1⅜		371	557	516	773		274	411	381	571
1½		373	559	518	776		275	413	383	574
1⅝		374	561	519	779		277	415	384	577
1¾		375	563	521	782		278	417	386	579
1⅞		377	565	523	785		279	419	388	582
2		378	567	525	788		281	421	390	585
2¼		381	571	529	793		284	425	394	591
2½		383	575	533	799		286	429	398	596
2¾		386	579	536	804		289	433	401	602
3		389	583	540	810		292	437	405	608
1¼	11	337	506	469	703	8	240	360	334	501
1⅜		339	508	471	706		242	362	336	503
1½		340	510	473	709		243	364	338	506
1⅝		342	512	474	712		244	367	339	509
1¾		343	514	476	714		246	369	341	512
1⅞		344	516	478	717		247	371	343	515
2		346	518	480	720		248	373	345	518
2¼		348	522	484	726		251	377	349	523
2½		351	526	488	731		254	381	353	529
2¾		354	531	491	737		257	385	356	534
3		356	535	495	743		259	389	360	540
1¼	10	305	458	424	636	7	208	312	289	433
1⅜		306	460	426	638		209	314	291	436
1½		308	462	428	641		211	316	293	439
1⅝		309	464	429	644		212	318	294	442
1¾		310	466	431	647		213	320	296	444
1⅞		312	468	433	650		215	322	298	447
2		313	470	435	653		216	324	300	450
2¼		316	474	439	658		219	328	304	456
2½		319	478	443	664		221	332	308	461
2¾		321	482	446	669		224	336	311	467
3		324	486	450	675		227	340	315	473

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 9–3b (continued)
Block Shear Shear Yielding Component
per inch of thickness, kips/in.



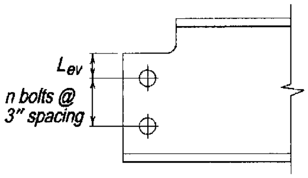
L_{ev}, in.	n	F_y, ksi				n	F_y, ksi			
		36		50			36		50	
		$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$	$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$		$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$	$\frac{0.6F_yA_{gv}}{t\Omega}$	$\frac{\phi 0.6F_yA_{gv}}{t}$
		ASD	LRFD	ASD	LRFD		ASD	LRFD	ASD	LRFD
1¼	6	175	263	244	366	3	78.3	117	109	163
1⅜		177	265	246	368		79.6	119	111	166
1½		178	267	248	371		81.0	121	113	169
1⅝		180	269	249	374		82.3	124	114	172
1¾		181	271	251	377		83.7	126	116	174
1⅞		182	273	253	380		85.0	128	118	177
2		184	275	255	383		86.4	130	120	180
2¼		186	279	259	388		89.1	134	124	186
2½		189	283	263	394		91.8	138	128	191
2¾		192	288	266	399		94.5	142	131	197
3		194	292	270	405		97.2	146	135	203
1¼	5	143	215	199	298	2	45.9	68.8	63.8	95.6
1⅜		144	217	201	301		47.2	70.9	65.6	98.4
1½		146	219	203	304		48.6	72.9	67.5	101
1⅝		147	221	204	307		49.9	74.9	69.4	104
1¾		148	223	206	309		51.3	76.9	71.3	107
1⅞		150	225	208	312		52.7	79.0	73.1	110
2		151	227	210	315		54.0	81.0	75.0	113
2¼		154	231	214	321		56.7	85.0	78.8	118
2½		157	235	218	326		59.4	89.1	82.5	124
2¾		159	239	221	332		62.1	93.1	86.3	129
3		162	243	225	338		64.8	97.2	90.0	135
1¼	4	111	166	154	231					
1⅜		112	168	156	233					
1½		113	170	158	236					
1⅝		115	172	159	239					
1¾		116	174	161	242					
1⅞		117	176	163	245					
2		119	178	165	248					
2¼		121	182	169	253					
2½		124	186	173	259					
2¾		127	190	176	264					
3		130	194	180	270					

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Table 9–3c

Block Shear Shear Rupture Component

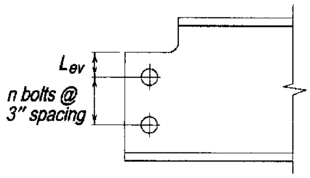
per inch of thickness, kips/in.



n	L_{ev}, in.	F_u, ksi											
		58						65					
		3/4		7/8		1		3/4		7/8		1	
		$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
12	1¼	**421**	631	**396**	594	**371**	556	**472**	707	**444**	665	**416**	623
	1⅜	**423**	635	**398**	597	**373**	560	**474**	711	**446**	669	**418**	627
	1½	**425**	638	**400**	600	**375**	563	**477**	715	**449**	673	**420**	631
	1⅝	**427**	641	**402**	604	**377**	566	**479**	718	**451**	676	**423**	634
	1¾	**430**	644	**405**	607	**380**	569	**481**	722	**453**	680	**425**	638
	1⅞	**432**	648	**407**	610	**382**	573	**484**	726	**456**	684	**428**	642
	2	**434**	651	**409**	613	**384**	576	**486**	729	**458**	687	**430**	645
	2¼	**438**	657	**413**	620	**388**	582	**491**	737	**463**	695	**435**	653
	2½	**443**	664	**418**	626	**393**	589	**496**	744	**468**	702	**440**	660
	2¾	**447**	670	**422**	633	**397**	595	**501**	751	**473**	709	**445**	667
	3	**451**	677	**426**	639	**401**	602	**506**	759	**478**	717	**450**	675
11	1¼	**384**	576	**361**	542	**338**	507	**430**	645	**405**	607	**379**	569
	1⅜	**386**	579	**363**	545	**340**	511	**433**	649	**407**	611	**381**	572
	1½	**388**	582	**365**	548	**343**	514	**435**	653	**410**	614	**384**	576
	1⅝	**390**	586	**368**	551	**345**	517	**438**	656	**412**	618	**386**	580
	1¾	**393**	589	**370**	555	**347**	520	**440**	660	**414**	622	**389**	583
	1⅞	**395**	592	**372**	558	**349**	524	**442**	664	**417**	625	**391**	587
	2	**397**	595	**374**	561	**351**	527	**445**	667	**419**	629	**394**	590
	2¼	**401**	602	**378**	568	**356**	533	**450**	675	**424**	636	**399**	598
	2½	**406**	608	**383**	574	**360**	540	**455**	682	**429**	644	**403**	605
	2¾	**410**	615	**387**	581	**364**	546	**459**	689	**434**	651	**408**	612
	3	**414**	622	**391**	587	**369**	553	**464**	697	**439**	658	**413**	620
10	1¼	**347**	520	**326**	489	**306**	458	**389**	583	**366**	548	**342**	514
	1⅜	**349**	524	**328**	493	**308**	462	**391**	587	**368**	552	**345**	517
	1½	**351**	527	**331**	496	**310**	465	**394**	590	**371**	556	**347**	521
	1⅝	**353**	530	**333**	499	**312**	468	**396**	594	**373**	559	**350**	525
	1¾	**356**	533	**335**	502	**314**	471	**399**	598	**375**	563	**352**	528
	1⅞	**358**	537	**337**	506	**316**	475	**401**	601	**378**	567	**355**	532
	2	**360**	540	**339**	509	**319**	478	**403**	605	**380**	570	**357**	536
	2¼	**364**	546	**344**	515	**323**	484	**408**	612	**385**	578	**362**	543
	2½	**369**	553	**348**	522	**327**	491	**413**	620	**390**	585	**367**	550
	2¾	**373**	560	**352**	529	**332**	498	**418**	627	**395**	592	**372**	558
	3	**377**	566	**357**	535	**336**	504	**423**	634	**400**	600	**377**	565

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

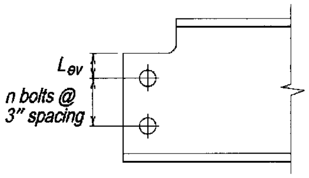
Table 9–3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kips/in.



n	L_{ev}, in.	F_u, ksi											
		58						65					
		3/4		7/8		1		3/4		7/8		1	
		$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
9	1¼	**310**	465	**291**	437	**273**	409	**347**	521	**327**	490	**306**	459
	1⅜	**312**	468	**294**	440	**275**	413	**350**	525	**329**	494	**308**	463
	1½	**314**	471	**296**	444	**277**	416	**352**	528	**332**	497	**311**	466
	1⅝	**316**	475	**298**	447	**279**	419	**355**	532	**334**	501	**313**	470
	1¾	**319**	478	**300**	450	**282**	422	**357**	536	**336**	505	**316**	473
	1⅞	**321**	481	**302**	453	**284**	426	**360**	539	**339**	508	**318**	477
	2	**323**	484	**305**	457	**286**	429	**362**	543	**341**	512	**321**	481
	2¼	**327**	491	**309**	463	**290**	436	**367**	550	**346**	519	**325**	488
	2½	**332**	498	**313**	470	**295**	442	**372**	558	**351**	527	**330**	495
	2¾	**336**	504	**318**	476	**299**	449	**377**	565	**356**	534	**335**	503
	3	**340**	511	**322**	483	**303**	455	**381**	572	**361**	541	**340**	510
8	1¼	**273**	409	**257**	385	**240**	361	**306**	459	**288**	431	**269**	404
	1⅜	**275**	413	**259**	388	**243**	364	**308**	463	**290**	435	**272**	408
	1½	**277**	416	**261**	392	**245**	367	**311**	466	**293**	439	**274**	411
	1⅝	**279**	419	**263**	395	**247**	370	**313**	470	**295**	442	**277**	415
	1¾	**282**	422	**265**	398	**249**	374	**316**	473	**297**	446	**279**	419
	1⅞	**284**	426	**268**	401	**251**	377	**318**	477	**300**	450	**282**	422
	2	**286**	429	**270**	405	**253**	380	**321**	481	**302**	453	**284**	426
	2¼	**290**	436	**274**	411	**258**	387	**325**	488	**307**	461	**289**	433
	2½	**295**	442	**278**	418	**262**	393	**330**	495	**312**	468	**294**	441
	2¾	**299**	449	**283**	424	**266**	400	**335**	503	**317**	475	**299**	448
	3	**303**	455	**287**	431	**271**	406	**340**	510	**322**	483	**303**	455
7	1¼	**236**	354	**222**	333	**208**	312	**264**	397	**249**	373	**233**	349
	1⅜	**238**	357	**224**	336	**210**	315	**267**	400	**251**	377	**235**	353
	1½	**240**	361	**226**	339	**212**	318	**269**	404	**254**	380	**238**	356
	1⅝	**243**	364	**228**	343	**214**	321	**272**	408	**256**	384	**240**	360
	1¾	**245**	367	**231**	346	**216**	325	**274**	411	**258**	388	**243**	364
	1⅞	**247**	370	**233**	349	**219**	328	**277**	415	**261**	391	**245**	367
	2	**249**	374	**235**	352	**221**	331	**279**	419	**263**	395	**247**	371
	2¼	**253**	380	**239**	359	**225**	338	**284**	426	**268**	402	**252**	378
	2½	**258**	387	**244**	365	**229**	344	**289**	433	**273**	410	**257**	386
	2¾	**262**	393	**248**	372	**234**	351	**294**	441	**278**	417	**262**	393
	3	**266**	400	**252**	378	**238**	357	**299**	448	**283**	424	**267**	400

ASD	**LRFD**
$\Omega = 2.00$	$\phi = 0.75$

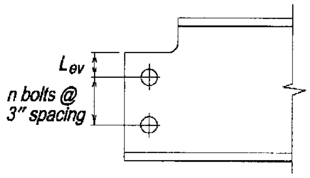
Table 9–3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kips/in.



n	L_{ev}, in.	F_u, ksi											
		58						65					
		$^3/_4$		$^7/_8$		1		$^3/_4$		$^7/_8$		1	
		$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
6	**1¼**	199	299	187	281	175	263	223	335	210	314	196	294
	1⅜	201	302	189	284	177	266	225	338	212	318	199	298
	1½	203	305	191	287	179	269	228	342	215	322	201	302
	1⅝	206	308	194	290	182	272	230	346	217	325	204	305
	1¾	208	312	196	294	184	276	233	349	219	329	206	309
	1⅞	210	315	198	297	186	279	235	353	222	333	208	313
	2	212	318	200	300	188	282	238	356	224	336	211	316
	2¼	216	325	204	307	192	289	243	364	229	344	216	324
	2½	221	331	209	313	197	295	247	371	234	351	221	331
	2¾	225	338	213	320	201	302	252	378	239	358	225	338
	3	229	344	217	326	206	308	257	386	244	366	230	346
5	**1¼**	162	243	152	228	142	214	182	272	171	256	160	239
	1⅜	164	246	154	232	145	217	184	276	173	260	162	243
	1½	166	250	157	235	147	220	186	280	176	263	165	247
	1⅝	169	253	159	238	149	223	189	283	178	267	167	250
	1¾	171	256	161	241	151	227	191	287	180	271	169	254
	1⅞	173	259	163	245	153	230	194	291	183	274	172	258
	2	175	263	165	248	156	233	196	294	185	278	174	261
	2¼	179	269	170	254	160	240	201	302	190	285	179	269
	2½	184	276	174	261	164	246	206	309	195	293	184	276
	2¾	188	282	178	268	169	253	211	316	200	300	189	283
	3	192	289	183	274	173	259	216	324	205	307	194	291
4	**1¼**	125	188	117	176	110	165	140	210	132	197	123	185
	1⅜	127	191	120	179	112	168	143	214	134	201	126	188
	1½	129	194	122	183	114	171	145	218	137	205	128	192
	1⅝	132	197	124	186	116	175	147	221	139	208	130	196
	1¾	134	201	126	189	119	178	150	225	141	212	133	199
	1⅞	136	204	128	192	121	181	152	229	144	216	135	203
	2	138	207	131	196	123	184	155	232	146	219	138	207
	2¼	142	214	135	202	127	191	160	239	151	227	143	214
	2½	147	220	139	209	132	197	165	247	156	234	147	221
	2¾	151	227	144	215	136	204	169	254	161	241	152	229
	3	156	233	148	222	140	210	174	261	166	249	157	236

ASD	**LRFD**
$\Omega = 2.00$	$\phi = 0.75$

Table 9–3c (continued)
Block Shear
Shear Rupture Component
per inch of thickness, kips/in.



n	L_{ev}, in.	F_u, ksi											
		58						65					
		3/4		7/8		1		3/4		7/8		1	
		$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$	$\frac{0.6F_uA_{nv}}{t\Omega}$	$\frac{\phi 0.6F_uA_{nv}}{t}$
		ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
3	1¼	88.1	132	82.6	124	77.2	116	98.7	148	92.6	139	86.5	130
	1⅜	90.3	135	84.8	127	79.4	119	101	152	95.1	143	89.0	133
	1½	92.4	139	87.0	131	81.6	122	104	155	97.5	146	91.4	137
	1⅝	94.6	142	89.2	134	83.7	126	106	159	99.9	150	93.8	141
	1¾	96.8	145	91.4	137	85.9	129	108	163	102	154	96.3	144
	1⅞	99.0	148	93.5	140	88.1	132	111	166	105	157	98.7	148
	2	101	152	95.7	144	90.3	135	113	170	107	161	101	152
	2¼	105	158	100	150	94.6	142	118	177	112	168	106	159
	2½	110	165	104	157	99.0	148	123	185	117	176	111	166
	2¾	114	171	109	163	103	155	128	192	122	183	116	174
	3	119	178	113	170	108	161	133	199	127	190	121	181
2	1¼	51.1	76.7	47.8	71.8	44.6	66.9	57.3	85.9	53.6	80.4	50.0	75.0
	1⅜	53.3	79.9	50.0	75.0	46.8	70.1	59.7	89.6	56.1	84.1	52.4	78.6
	1½	55.5	83.2	52.2	78.3	48.9	73.4	62.2	93.2	58.5	87.8	54.8	82.3
	1⅝	57.6	86.5	54.4	81.6	51.1	76.7	64.6	96.9	60.9	91.4	57.3	85.9
	1¾	59.8	89.7	56.6	84.8	53.3	79.9	67.0	101	63.4	95.1	59.7	89.6
	1⅞	62.0	93.0	58.7	88.1	55.5	83.2	69.5	104	65.8	98.7	62.2	93.2
	2	64.2	96.2	60.9	91.4	57.6	86.5	71.9	108	68.3	102	64.6	96.9
	2¼	68.5	103	65.3	97.9	62.0	93.0	76.8	115	73.1	110	69.5	104
	2½	72.9	109	69.6	104	66.3	99.5	81.7	122	78.0	117	74.3	112
	2¾	77.2	116	73.9	111	70.7	106	86.5	130	82.9	124	79.2	119
	3	81.6	122	78.3	117	75.0	113	91.4	137	87.8	132	84.1	126

ASD	**LRFD**
$\Omega = 2.00$	$\phi = 0.75$

F_y = 50 ksi

Table 9–4
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W44×335	218	328	34.2	51.3	332	499	9.99	15.0
×290	170	255	28.8	43.3	243	365	6.81	10.2
×262	144	216	26.2	39.3	199	299	5.70	8.55
×230	119	178	23.7	35.5	159	239	4.94	7.41
W40×593	658	987	59.7	89.5	1040	1550	29.8	44.8
×503	503	755	51.2	76.8	760	1140	22.6	33.8
×431	395	593	44.7	67.0	574	861	17.9	26.8
×397	344	516	40.7	61.0	481	722	14.6	21.8
×372	312	468	38.7	58.0	430	645	13.6	20.4
×362	298	447	37.3	56.0	405	607	12.5	18.7
×324	249	374	33.3	50.0	324	486	9.94	14.9
×297	219	329	31.0	46.5	277	416	8.84	13.3
×277	191	286	27.7	41.5	229	343	6.61	9.91
×249	163	244	25.0	37.5	186	280	5.45	8.18
×215	130	195	21.7	32.5	139	209	4.17	6.26
×199	122	182	21.7	32.5	130	195	4.82	7.23
W40×392	436	655	47.2	70.8	644	965	19.5	29.3
×331	336	504	40.7	61.0	473	710	15.1	22.7
×327	326	488	39.3	59.0	451	676	13.7	20.5
×294	275	412	35.3	53.0	365	548	11.0	16.6
×278	255	383	34.2	51.3	336	504	10.7	16.1
×264	233	349	32.0	48.0	298	447	9.24	13.9
×235	191	286	27.7	41.5	229	343	6.61	9.91
×211	162	243	25.0	37.5	186	279	5.47	8.21
×183	129	193	21.7	32.5	138	207	4.24	6.36
×167	119	179	21.7	32.5	128	192	5.02	7.52
×149	106	158	21.0	31.5	110	165	5.70	8.55
W36×800	1040	1560	79.3	119	1830	2750	53.4	80.0
×652	737	1110	65.7	98.5	1250	1880	38.0	57.0
×529	518	777	53.7	80.5	839	1260	26.0	39.1
×487	454	680	50.0	75.0	724	1090	23.1	34.7
×441	384	576	45.3	68.0	597	895	19.2	28.8
×395	320	480	40.7	61.0	481	722	15.5	23.3
×361	276	414	37.3	56.0	405	607	13.3	19.9
×330	238	357	34.0	51.0	337	506	11.0	16.5
×302	207	311	31.5	47.3	287	430	9.72	14.6
×282	186	279	29.5	44.3	251	377	8.60	12.9
×262	167	251	28.0	42.0	222	334	8.07	12.1
×247	153	230	26.7	40.0	200	300	7.47	11.2
×231	140	210	25.3	38.0	179	269	6.90	10.3

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nom-inal Wt.	R_5		R_6		$(N = 3^1/_4)$						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
335	303	455	13.3	20.0	329	495	329	495	547	823	902	1350
290	223	335	9.08	13.6	264	396	264	396	434	651	755	1130
262	183	274	7.60	11.4	218	327	229	344	373	560	680	1020
230	145	218	6.59	9.88	175	263	196	293	315	471	547	823
593	951	1430	39.8	59.7	—	—	—	—	1510	2260	1540	2310
503	697	1050	30.1	45.1	—	—	—	—	1170	1760	1290	1940
431	525	787	23.8	35.7	—	—	—	—	935	1400	1110	1660
397	442	662	19.4	29.1	—	—	—	—	820	1230	999	1500
372	394	590	18.1	27.1	438	657	438	657	750	1120	943	1410
362	371	557	16.6	24.9	419	629	419	629	717	1080	908	1360
324	297	446	13.3	19.9	356	534	357	537	606	911	803	1200
297	254	381	11.8	17.7	306	459	320	480	539	809	741	1110
277	211	317	8.81	13.2	250	375	281	421	472	707	659	988
249	172	258	7.27	10.9	204	307	244	366	407	610	591	886
215	129	193	5.56	8.34	153	229	201	301	305	459	507	760
199	118	177	6.42	9.64	146	218	193	288	291	437	503	754
392	589	884	26.1	39.1	—	—	—	—	1030	1540	1180	1760
331	432	648	20.2	30.3	—	—	—	—	804	1210	995	1490
327	413	620	18.2	27.3	—	—	—	—	780	1170	963	1440
294	335	503	14.7	22.1	390	584	390	584	665	996	856	1280
278	308	461	14.3	21.4	366	550	366	550	621	933	823	1230
264	273	410	12.3	18.5	328	492	337	505	570	854	768	1150
235	211	317	8.81	13.2	250	375	281	421	472	707	659	988
211	172	258	7.30	10.9	204	306	243	365	405	608	591	886
183	127	191	5.66	8.48	152	228	200	299	304	455	507	760
167	115	172	6.69	10.0	144	216	190	285	289	433	502	753
149	95.2	143	7.60	11.4	129	193	174	260	257	386	432	650
800	1680	2520	71.1	107	—	—	—	—	2340	3510	2030	3040
652	1150	1720	50.7	76.0	—	—	—	—	1690	2540	1620	2430
529	770	1160	34.7	52.1	—	—	—	—	1210	1820	1280	1920
487	664	995	30.8	46.3	—	—	—	—	1070	1600	1180	1770
441	547	820	25.6	38.3	—	—	—	—	915	1370	1060	1590
395	442	662	20.7	31.1	452	678	452	678	772	1160	937	1410
361	371	557	17.7	26.6	397	596	397	596	673	1010	851	1280
330	310	465	14.7	22.0	349	523	349	523	587	880	768	1150
302	263	394	13.0	19.4	309	465	309	465	516	776	706	1060
282	230	345	11.5	17.2	279	419	282	423	468	702	657	985
262	203	304	10.8	16.1	248	373	258	388	425	639	619	929
247	182	273	9.96	14.9	224	336	240	360	393	590	587	880
231	162	243	9.20	13.8	201	302	222	334	362	544	555	832

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued)
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W36×256	199	298	32.0	48.0	298	447	9.87	14.8
×232	168	253	29.0	43.5	245	367	8.16	12.2
×210	146	219	27.7	41.5	212	319	8.28	12.4
×194	128	192	25.5	38.3	181	271	7.04	10.6
×182	117	175	24.2	36.3	161	242	6.42	9.63
×170	105	157	22.7	34.0	142	212	5.71	8.56
×160	96.0	144	21.7	32.5	127	191	5.40	8.10
×150	88.1	132	20.8	31.3	115	173	5.23	7.85
×135	77.1	116	20.0	30.0	99.5	149	5.56	8.34
W33×387	322	483	42.0	63.0	514	771	17.6	26.4
×354	278	417	38.7	58.0	435	652	15.2	22.8
×318	232	348	34.7	52.0	351	527	12.2	18.3
×291	201	302	32.0	48.0	298	447	10.6	15.9
×263	171	256	29.0	43.5	245	367	8.78	13.2
×241	151	227	27.7	41.5	215	323	8.63	12.9
×221	133	200	25.8	38.8	186	278	7.77	11.7
×201	115	173	23.8	35.8	156	234	6.82	10.2
W33×169	107	161	22.3	33.5	146	219	5.27	7.90
×152	92.9	139	21.2	31.8	125	188	5.24	7.85
×141	83.7	126	20.2	30.3	111	167	5.00	7.51
×130	75.2	113	19.3	29.0	98.4	148	4.98	7.47
×118	66.0	99.1	18.3	27.5	84.5	127	4.94	7.42
W30×391	366	549	45.3	68.0	597	895	22.4	33.7
×357	313	469	41.3	62.0	498	747	18.7	28.1
×326	270	404	38.0	57.0	420	630	16.1	24.2
×292	224	336	34.0	51.0	337	506	12.9	19.4
×261	189	283	31.0	46.5	277	416	11.1	16.7
×235	158	237	27.7	41.5	223	335	8.80	13.2
×211	136	204	25.8	38.8	188	283	8.27	12.4
×191	117	175	23.7	35.5	157	235	7.11	10.7
×173	101	152	21.8	32.8	132	198	6.26	9.39
W30×148	99.1	149	21.7	32.5	137	206	5.48	8.22
×132	84.6	127	20.5	30.8	116	174	5.54	8.32
×124	77.0	116	19.5	29.3	104	156	5.16	7.73
×116	70.6	106	18.8	28.3	94.3	141	5.11	7.66
×108	64.0	96.1	18.2	27.3	84.5	127	5.16	7.74
×99	57.2	85.8	17.3	26.0	73.9	111	5.11	7.67
×90	49.3	74.0	15.7	23.5	60.6	90.9	4.16	6.25

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

$F_y = 50$ ksi

Nominal Wt.	R_5		R_6		(N = 3¹/₄)						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
256	273	410	13.2	19.7	303	454	303	454	502	752	719	1080
232	225	337	10.9	16.3	262	394	262	394	430	647	646	969
210	192	288	11.0	16.6	236	354	236	354	382	573	609	914
194	164	246	9.38	14.1	204	305	211	316	339	508	558	837
182	146	219	8.56	12.8	182	273	196	293	313	468	527	790
170	128	192	7.61	11.4	161	240	179	268	284	425	492	738
160	114	172	7.20	10.8	145	217	167	250	263	394	468	702
150	103	154	6.98	10.5	132	199	156	234	244	366	448	672
135	86.3	129	7.41	11.1	118	176	142	214	219	330	383	576
387	472	708	23.5	35.3	459	688	459	688	781	1170	906	1360
354	399	599	20.2	30.4	404	606	404	606	682	1020	825	1240
318	322	484	16.3	24.5	345	517	345	517	577	865	731	1100
291	273	410	14.1	21.2	305	458	305	458	506	760	669	1000
263	225	337	11.7	17.6	265	397	265	397	436	653	601	901
241	196	294	11.5	17.3	241	362	241	362	392	589	567	851
221	168	252	10.4	15.5	211	316	217	326	350	526	526	789
201	141	211	9.09	13.6	178	267	192	289	307	462	482	722
169	134	201	7.02	10.5	163	245	179	270	286	431	453	680
152	113	170	6.98	10.5	142	214	162	242	255	381	425	638
141	99.9	150	6.67	10.0	127	191	149	224	233	350	403	604
130	87.4	131	6.64	9.97	115	172	138	207	213	320	384	576
118	73.7	111	6.59	9.89	101	151	125	188	191	288	325	488
391	547	820	29.9	44.9	513	770	513	770	879	1320	903	1350
357	457	685	25.0	37.5	447	671	447	671	760	1140	813	1220
326	385	577	21.5	32.2	394	589	394	589	664	993	739	1110
292	310	465	17.3	25.9	335	502	335	502	559	838	653	980
261	254	381	14.9	22.3	290	434	290	434	479	717	588	882
235	205	307	11.7	17.6	248	372	248	372	406	609	520	779
211	171	257	11.0	16.5	215	323	220	330	356	534	480	719
191	142	213	9.48	14.2	180	270	194	290	311	465	436	653
173	119	179	8.35	12.5	152	229	172	259	273	411	399	598
148	126	189	7.31	11.0	155	233	170	255	269	404	399	598
132	105	157	7.39	11.1	134	201	151	227	236	354	373	559
124	93.5	140	6.87	10.3	121	181	140	211	217	327	353	529
116	84.1	126	6.81	10.2	111	166	132	198	202	304	339	509
108	74.2	111	6.88	10.3	101	152	123	185	187	281	325	488
99	63.8	95.7	6.82	10.2	90.5	136	113	170	171	256	308	463
90	52.4	78.6	5.55	8.33	74.1	111	100	150	148	222	249	375

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

Table 9–4 (continued)

F_y = 50 ksi

Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W27×539	**710**	1070	**65.7**	98.5	**1250**	1880	**48.0**	72.0
×368	**376**	564	**46.0**	69.0	**615**	922	**25.2**	37.8
×336	**322**	483	**42.0**	63.0	**514**	771	**21.1**	31.7
×307	**278**	417	**38.7**	58.0	**435**	652	**18.2**	27.3
×281	**240**	360	**35.3**	53.0	**365**	548	**15.2**	22.8
×258	**209**	313	**32.7**	49.0	**311**	466	**13.3**	19.9
×235	**182**	273	**30.3**	45.5	**265**	398	**11.8**	17.7
×217	**158**	237	**27.7**	41.5	**223**	335	**9.69**	14.5
×194	**133**	199	**25.0**	37.5	**181**	272	**8.09**	12.1
×178	**119**	179	**24.2**	36.3	**162**	243	**8.32**	12.5
×161	**103**	154	**22.0**	33.0	**134**	201	**6.97**	10.5
×146	**88.8**	133	**20.2**	30.3	**112**	168	**5.99**	8.99
W27×129	**86.3**	129	**20.3**	30.5	**120**	181	**5.40**	8.09
×114	**72.6**	109	**19.0**	28.5	**99.9**	150	**5.27**	7.91
×102	**61.3**	91.9	**17.2**	25.8	**81.1**	122	**4.39**	6.58
×94	**54.8**	82.3	**16.3**	24.5	**71.3**	107	**4.24**	6.36
×84	**47.5**	71.2	**15.3**	23.0	**60.1**	90.2	**4.11**	6.17
W24×370	**408**	612	**50.7**	76.0	**744**	1120	**33.3**	50.0
×335	**343**	514	**46.0**	69.0	**615**	922	**27.8**	41.7
×306	**292**	438	**42.0**	63.0	**514**	771	**23.4**	35.0
×279	**250**	376	**38.7**	58.0	**435**	652	**20.2**	30.3
×250	**207**	311	**34.7**	52.0	**351**	527	**16.3**	24.5
×229	**178**	268	**32.0**	48.0	**298**	447	**14.2**	21.3
×207	**150**	225	**29.0**	43.5	**245**	367	**11.8**	17.7
×192	**132**	198	**27.0**	40.5	**212**	318	**10.3**	15.5
×176	**115**	173	**25.0**	37.5	**181**	272	**9.01**	13.5
×162	**101**	152	**23.5**	35.3	**157**	236	**8.30**	12.5
×146	**86.1**	129	**21.7**	32.5	**132**	198	**7.36**	11.0
×131	**73.6**	110	**20.2**	30.3	**111**	167	**6.81**	10.2
×117	**61.9**	92.8	**18.3**	27.5	**90.6**	136	**5.83**	8.74
×104	**52.1**	78.1	**16.7**	25.0	**73.7**	111	**5.00**	7.51
W24×103	**67.8**	102	**18.3**	27.5	**97.2**	146	**5.00**	7.50
×94	**59.0**	88.5	**17.2**	25.8	**83.3**	125	**4.64**	6.96
×84	**49.7**	74.6	**15.7**	23.5	**68.1**	102	**4.04**	6.06
×76	**43.3**	64.9	**14.7**	22.0	**58.0**	86.9	**3.78**	5.68
×68	**37.5**	56.3	**13.8**	20.8	**49.2**	73.9	**3.72**	5.58
×62	**39.1**	58.6	**14.3**	21.5	**52.2**	78.2	**4.10**	6.15
×55	**33.1**	49.6	**13.2**	19.8	**42.5**	63.7	**3.74**	5.61

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

$F_y = 50$ ksi

Nom-inal Wt.	R_5		R_6		($N = 3^1/_4$)						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
539	1150	1720	64.0	96.0	—	—	—	—	1630	2460	1280	1920
368	564	846	33.6	50.4	—	—	—	—	902	1350	839	1260
336	472	708	28.2	42.3	459	688	459	688	781	1170	756	1130
307	399	599	24.3	36.4	404	606	404	606	682	1020	687	1030
281	335	503	20.3	30.4	355	532	355	532	595	892	621	931
258	285	428	17.7	26.5	315	472	315	472	524	785	568	852
235	243	364	15.7	23.6	280	421	280	421	462	694	522	782
217	205	307	12.9	19.4	248	372	248	372	406	609	472	708
194	166	249	10.8	16.2	207	311	214	321	347	520	422	632
178	147	220	11.1	16.6	189	284	198	297	317	476	403	605
161	121	182	9.29	13.9	157	235	175	261	278	415	364	546
146	101	151	7.99	12.0	131	197	154	231	243	364	331	497
129	110	166	7.19	10.8	138	207	152	228	239	357	337	506
114	90.4	136	7.03	10.5	117	176	134	202	207	311	311	467
102	73.2	110	5.85	8.78	95.4	143	117	176	179	268	279	419
94	63.7	95.5	5.65	8.48	85.1	128	108	162	163	244	264	396
84	52.8	79.2	5.49	8.23	73.5	110	97.2	146	145	217	246	369
370	682	1020	44.4	66.7	573	859	573	859	981	1470	851	1280
335	564	846	37.1	55.6	493	738	493	738	836	1250	760	1140
306	472	708	31.2	46.7	429	643	429	643	721	1080	684	1030
279	399	599	26.9	40.4	376	565	376	565	626	941	620	930
250	322	484	21.8	32.7	320	480	320	480	527	791	548	822
229	273	410	18.9	28.4	282	424	282	424	460	692	500	749
207	225	337	15.7	23.6	244	366	244	366	394	591	447	671
192	195	292	13.8	20.7	220	330	220	330	352	528	413	619
176	166	249	12.0	18.0	196	295	196	295	311	468	379	568
162	144	215	11.1	16.6	177	267	177	267	278	419	353	529
146	120	179	9.81	14.7	156	234	157	235	243	364	322	482
131	99.9	150	9.08	13.6	133	200	139	208	213	318	296	444
117	81.1	122	7.77	11.7	110	164	121	182	183	275	267	400
104	65.7	98.6	6.67	10.0	90.0	135	106	159	158	237	241	361
103	89.1	134	6.67	10.0	113	170	127	191	195	293	270	405
94	75.7	114	6.19	9.28	98.4	148	115	172	174	261	250	376
84	61.6	92.4	5.39	8.08	81.2	122	101	151	150	226	227	340
76	51.9	77.9	5.05	7.57	70.3	105	91.1	136	134	201	210	316
68	43.4	65.0	4.96	7.44	61.3	92.0	82.3	124	120	180	197	295
62	45.7	68.5	5.47	8.20	65.5	98.2	85.6	128	125	187	204	306
55	36.6	54.9	4.99	7.48	54.7	81.9	76.0	114	109	164	167	251

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued)
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W21×201	162	242	30.3	45.5	267	400	14.5	21.8
×182	137	205	27.7	41.5	222	332	12.3	18.4
×166	116	174	25.0	37.5	182	274	9.97	15.0
×147	99.0	149	24.0	36.0	158	237	10.6	15.9
×132	83.1	125	21.7	32.5	128	193	8.78	13.2
×122	73.0	110	20.0	30.0	110	165	7.50	11.2
×111	63.0	94.5	18.3	27.5	91.9	138	6.39	9.58
×101	54.2	81.3	16.7	25.0	76.2	114	5.29	7.93
W21×93	69.1	104	19.3	29.0	103	154	7.01	10.5
×83	57.3	85.9	17.2	25.8	81.3	122	5.52	8.27
×73	47.0	70.5	15.2	22.8	63.6	95.4	4.33	6.49
×68	42.5	63.7	14.3	21.5	56.2	84.3	3.97	5.95
×62	37.2	55.8	13.3	20.0	47.8	71.7	3.58	5.37
×55	31.9	47.9	12.5	18.8	40.0	59.9	3.51	5.26
×48	27.1	40.7	11.7	17.5	32.7	49.1	3.49	5.24
W21×57	38.8	58.2	13.5	20.3	50.0	75.1	3.51	5.26
×50	32.8	49.2	12.7	19.0	41.3	61.9	3.56	5.34
×44	27.7	41.6	11.7	17.5	33.5	50.2	3.33	5.00
W18×311	410	616	50.7	76.0	747	1120	41.5	62.2
×283	350	525	46.7	70.0	631	946	36.3	54.4
×258	288	432	42.7	64.0	529	793	30.7	46.0
×234	243	364	38.7	58.0	437	656	25.4	38.1
×211	204	306	35.3	53.0	363	545	21.8	32.7
×192	172	258	32.0	48.0	300	450	17.9	26.9
×175	148	222	29.7	44.5	255	382	16.0	24.0
×158	124	187	27.0	40.5	211	316	13.5	20.3
×143	105	157	24.3	36.5	173	259	10.9	16.4
×130	89.4	134	22.3	33.5	145	217	9.41	14.1
×119	79.8	120	21.8	32.8	131	197	10.1	15.1
×106	66.0	99.0	19.7	29.5	106	159	8.43	12.6
×97	56.7	85.1	17.8	26.8	87.9	132	6.84	10.3
×86	46.9	70.3	16.0	24.0	70.3	105	5.64	8.46
×76	38.3	57.5	14.2	21.3	55.0	82.5	4.48	6.72
W18×71	50.0	75.0	16.5	24.8	75.5	113	5.86	8.79
×65	43.2	64.8	15.0	22.5	63.0	94.4	4.78	7.18
×60	37.9	56.9	13.8	20.8	53.7	80.5	4.07	6.11
×55	33.5	50.3	13.0	19.5	46.6	69.8	3.76	5.63
×50	28.8	43.1	11.8	17.8	38.5	57.7	3.15	4.73

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nom-inal Wt.	R_5		R_6		($N = 3^1/_4$)						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
201	245	367	19.3	29.0	260	390	260	390	422	632	419	629
182	203	304	16.4	24.6	227	340	227	340	364	545	377	566
166	167	251	13.3	19.9	197	296	197	296	313	470	337	506
147	142	213	14.2	21.3	177	266	177	266	276	415	318	476
132	116	173	11.7	17.6	154	231	154	231	237	356	284	426
122	98.8	148	10.0	15.0	134	201	138	208	211	318	260	390
111	82.7	124	8.52	12.8	113	169	122	184	185	278	237	355
101	68.6	103	7.05	10.6	93.4	140	108	163	163	244	214	320
93	92.5	139	9.35	14.0	126	188	132	198	201	302	251	376
83	73.5	110	7.35	11.0	99.2	149	113	170	171	256	221	331
73	57.5	86.2	5.77	8.66	77.7	116	96.4	145	143	215	193	290
68	50.6	75.9	5.29	7.94	69.1	104	89.0	134	131	197	182	273
62	42.8	64.2	4.78	7.16	59.4	89.2	80.4	121	118	177	168	252
55	35.1	52.6	4.68	7.02	51.4	77.0	72.5	109	103	154	156	234
48	27.9	41.8	4.66	6.99	44.0	66.1	65.1	97.6	88.1	132	144	217
57	45.1	67.7	4.67	7.01	61.4	92.2	82.7	124	121	182	171	256
50	36.3	54.5	4.74	7.11	52.9	79.3	74.1	111	106	159	158	237
44	28.9	43.3	4.44	6.66	44.3	66.5	65.7	98.5	88.6	133	145	217
311	685	1030	55.3	83.0	575	863	575	863	985	1480	679	1020
283	578	867	48.4	72.6	502	753	502	753	852	1280	612	918
258	485	728	40.9	61.4	427	640	427	640	715	1070	549	824
234	401	602	33.8	50.8	369	553	369	553	612	917	489	733
211	333	500	29.1	43.6	319	478	319	478	523	784	438	657
192	275	413	23.9	35.9	276	414	276	414	448	672	391	586
175	234	350	21.3	32.0	245	367	245	367	393	589	357	535
158	193	289	18.0	27.0	212	319	212	319	336	506	319	479
143	158	238	14.6	21.8	184	276	184	276	289	433	285	427
130	133	199	12.5	18.8	162	243	162	243	251	377	258	387
119	119	178	13.5	20.2	151	227	151	227	230	347	249	373
106	95.3	143	11.2	16.9	130	195	130	195	196	294	221	332
97	79.4	119	9.12	13.7	110	165	115	172	171	257	199	298
86	63.4	95.0	7.52	11.3	88.6	132	98.9	148	146	219	177	265
76	49.6	74.4	5.97	8.96	69.6	104	84.4	127	123	184	155	232
71	68.3	102	7.81	11.7	94.5	142	104	156	154	231	183	274
65	57.1	85.7	6.38	9.57	78.5	118	92.0	138	135	203	165	248
60	48.7	73.1	5.43	8.15	66.9	100	82.8	125	121	181	151	227
55	42.0	63.0	5.01	7.51	58.8	88.1	75.8	114	109	164	141	212
50	34.7	52.0	4.20	6.30	48.7	73.1	67.1	101	95.9	144	128	192

N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued)
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W18×46	**30.2**	45.3	**12.0**	18.0	**40.5**	60.7	**3.09**	4.63
×40	**24.3**	36.5	**10.5**	15.8	**30.9**	46.3	**2.40**	3.60
×35	**20.7**	31.0	**10.0**	15.0	**25.8**	38.7	**2.59**	3.89
W16×100	**67.6**	101	**19.5**	29.3	**107**	160	**8.65**	13.0
×89	**55.9**	83.8	**17.5**	26.3	**85.7**	129	**7.13**	10.7
×77	**44.1**	66.1	**15.2**	22.8	**64.4**	96.7	**5.42**	8.13
×67	**35.1**	52.7	**13.2**	19.8	**48.8**	73.1	**4.10**	6.15
W16×57	**40.0**	60.0	**14.3**	21.5	**57.4**	86.1	**4.89**	7.33
×50	**32.7**	49.0	**12.7**	19.0	**44.8**	67.2	**3.87**	5.81
×45	**27.8**	41.7	**11.5**	17.3	**36.7**	55.0	**3.26**	4.88
×40	**23.1**	34.6	**10.2**	15.3	**28.8**	43.2	**2.54**	3.80
×36	**20.5**	30.7	**9.83**	14.8	**25.3**	38.0	**2.72**	4.08
W16×31	**19.3**	28.9	**9.17**	13.8	**23.0**	34.6	**2.15**	3.23
×26	**15.6**	23.3	**8.33**	12.5	**17.7**	26.5	**2.09**	3.13
W14×730	**1410**	2110	**102**	154	**2870**	4310	**190**	285
×665	**1210**	1810	**94.3**	142	**2440**	3660	**167**	251
×605	**1030**	1540	**86.5**	130	**2050**	3080	**145**	218
×550	**876**	1310	**79.3**	119	**1730**	2590	**126**	189
×500	**748**	1120	**73.0**	110	**1460**	2190	**111**	166
×455	**639**	959	**67.2**	101	**1230**	1850	**96.8**	145
×426	**568**	851	**62.5**	93.8	**1080**	1620	**84.0**	126
×398	**508**	762	**59.0**	88.5	**957**	1430	**77.0**	115
×370	**449**	674	**55.2**	82.8	**836**	1250	**68.7**	103
×342	**394**	591	**51.3**	77.0	**723**	1090	**60.9**	91.4
×311	**336**	504	**47.0**	70.5	**606**	909	**52.3**	78.5
×283	**287**	430	**43.0**	64.5	**508**	762	**44.8**	67.1
×257	**244**	365	**39.2**	58.8	**422**	633	**37.9**	56.8
×233	**207**	310	**35.7**	53.5	**350**	524	**32.1**	48.1
×211	**176**	264	**32.7**	49.0	**292**	438	**27.7**	41.6
×193	**151**	227	**29.7**	44.5	**243**	364	**22.8**	34.3
×176	**132**	198	**27.7**	41.5	**208**	313	**20.7**	31.1
×159	**111**	167	**24.8**	37.3	**169**	253	**16.8**	25.1
×145	**95.7**	143	**22.7**	34.0	**141**	211	**14.1**	21.2
W14×132	**87.5**	131	**21.5**	32.3	**127**	190	**12.8**	19.3
×120	**75.6**	113	**19.7**	29.5	**106**	159	**10.9**	16.4
×109	**63.8**	95.7	**17.5**	26.3	**85.0**	127	**8.49**	12.7
×99	**55.7**	83.5	**16.2**	24.3	**71.8**	108	**7.46**	11.2
×90	**48.0**	71.9	**14.7**	22.0	**59.2**	88.8	**6.18**	9.27

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5		R_6		($N = 3^1/_4$)						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
46	36.7	55.1	4.11	6.17	50.5	75.7	69.2	104	99.4	149	130	195
40	28.0	42.0	3.20	4.81	38.7	58.0	58.4	87.9	77.4	116	113	169
35	22.7	34.1	3.46	5.19	34.2	51.3	53.2	79.8	68.4	103	106	159
100	97.2	146	11.5	17.3	131	196	131	196	199	297	199	298
89	77.7	117	9.51	14.3	109	164	113	169	169	253	176	264
77	58.5	87.7	7.23	10.8	82.0	123	93.5	140	138	206	150	225
67	44.3	66.4	5.47	8.20	62.1	93.1	78.0	117	113	170	129	194
57	52.1	78.1	6.52	9.78	73.3	110	86.5	130	126	190	141	212
50	40.6	60.9	5.16	7.74	57.4	86.1	74.0	111	107	160	124	185
45	33.2	49.8	4.34	6.51	47.3	71.0	65.2	97.9	93.0	140	111	167
40	26.1	39.2	3.38	5.07	37.1	55.7	56.3	84.3	74.1	111	97.7	146
36	22.4	33.6	3.63	5.44	34.2	51.3	52.4	78.8	68.3	103	93.6	140
31	20.8	31.1	2.87	4.30	30.1	45.1	49.1	73.8	60.0	90.2	87.3	131
26	15.5	23.3	2.78	4.17	24.5	36.9	42.7	63.9	49.0	73.3	70.5	106
730	2590	3880	253	380	—	—	—	—	3150	4720	1380	2060
665	2200	3290	223	335	—	—	—	—	2730	4080	1220	1840
605	1850	2780	193	290	—	—	—	—	2340	3500	1090	1630
550	1560	2340	168	252	—	—	—	—	2010	3010	963	1450
500	1320	1970	147	221	—	—	—	—	1730	2600	858	1290
455	1110	1670	129	194	—	—	—	—	1500	2250	767	1150
426	973	1460	112	168	—	—	—	—	1340	2010	700	1050
398	863	1290	103	154	—	—	—	—	1210	1810	647	971
370	754	1130	91.6	137	—	—	—	—	1080	1620	593	890
342	652	978	81.2	122	561	841	561	841	955	1430	540	810
311	546	820	69.8	105	489	733	489	733	825	1240	483	724
283	458	687	59.7	89.5	427	640	427	640	714	1070	432	648
257	380	571	50.5	75.7	371	556	371	556	615	921	385	577
233	315	473	42.8	64.2	323	484	323	484	530	794	343	515
211	263	394	37.0	55.5	282	423	282	423	458	687	308	462
193	219	329	30.5	45.7	248	372	248	372	399	599	276	413
176	187	281	27.6	41.4	222	333	222	333	354	531	253	379
159	152	228	22.3	33.5	192	288	192	288	303	455	223	335
145	127	191	18.8	28.2	169	254	169	254	265	397	201	302
132	114	171	17.1	25.7	157	236	157	236	245	367	189	284
120	95.3	143	14.5	21.8	140	209	140	209	215	322	171	256
109	76.9	115	11.3	17.0	114	170	121	181	184	277	150	226
99	64.8	97.2	9.95	14.9	97.1	146	108	162	164	246	137	206
90	53.4	80.2	8.24	12.4	80.2	121	95.8	143	144	215	123	185

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W14×82	61.5	92.2	17.0	25.5	81.1	122	7.83	11.7
×74	51.6	77.4	15.0	22.5	64.4	96.6	5.92	8.88
×68	45.3	68.0	13.8	20.8	54.6	81.9	5.11	7.66
×61	38.6	57.9	12.5	18.8	44.4	66.6	4.25	6.38
W14×53	38.6	57.9	12.3	18.5	44.0	66.1	3.98	5.98
×48	33.6	50.4	11.3	17.0	36.8	55.2	3.46	5.19
×43	28.5	42.7	10.2	15.3	29.5	44.3	2.83	4.25
W14×38	23.6	35.5	10.3	15.5	29.8	44.7	2.96	4.45
×34	20.3	30.5	9.50	14.3	24.7	37.1	2.63	3.94
×30	17.7	26.5	9.00	13.5	21.0	31.4	2.67	4.00
W14×26	17.4	26.1	8.50	12.8	20.1	30.1	2.05	3.08
×22	14.1	21.1	7.67	11.5	15.4	23.1	1.91	2.86
W12×336	526	788	59.2	88.8	979	1470	81.3	122
×305	447	671	54.2	81.3	820	1230	70.2	105
×279	391	587	51.0	76.5	716	1070	66.1	99.1
×252	331	497	46.5	69.8	595	893	56.6	84.9
×230	286	429	42.8	64.3	505	757	49.2	73.8
×210	246	368	39.3	59.0	426	638	42.5	63.7
×190	206	309	35.3	53.0	346	519	34.5	51.7
×170	173	259	32.0	48.0	283	424	29.2	43.8
×152	145	217	29.0	43.5	231	347	24.8	37.2
×136	122	182	26.3	39.5	189	284	21.3	31.9
×120	101	151	23.7	35.5	151	227	17.8	26.8
×106	80.7	121	20.3	30.5	114	171	12.9	19.3
×96	68.7	103	18.3	27.5	93.2	140	10.5	15.8
×87	60.4	90.6	17.2	25.8	80.1	120	9.72	14.6
×79	52.2	78.3	15.7	23.5	66.5	99.8	8.24	12.4
×72	45.4	68.2	14.3	21.5	55.6	83.4	7.00	10.5
×65	39.1	58.6	13.0	19.5	45.6	68.4	5.84	8.77
W12×58	37.2	55.8	12.0	18.0	41.6	62.4	4.32	6.48
×53	33.8	50.7	11.5	17.3	37.0	55.5	4.28	6.42
W12×50	35.1	52.7	12.3	18.5	43.4	65.0	4.69	7.04
×45	30.0	45.0	11.2	16.8	35.4	53.1	3.92	5.88
×40	25.0	37.4	9.83	14.8	27.7	41.5	3.02	4.52
W12×35	20.5	30.7	10.0	15.0	28.5	42.8	3.00	4.50
×30	16.0	24.1	8.67	13.0	21.2	31.8	2.34	3.51
×26	13.0	19.6	7.67	11.5	16.4	24.6	1.89	2.84

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

F_y = 50 ksi

Nominal Wt.	R_5		R_6		(N = 3¼)						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
82	73.6	110	10.4	15.7	107	161	117	175	178	267	146	219
74	58.8	88.2	7.89	11.8	84.4	127	100	151	152	228	128	191
68	49.9	74.8	6.81	10.2	72.0	108	90.2	136	135	204	117	175
61	40.5	60.7	5.67	8.51	58.9	88.4	79.2	119	116	175	104	156
53	40.3	60.5	5.31	7.97	57.6	86.4	78.6	118	114	171	103	155
48	33.6	50.5	4.61	6.92	48.6	73.0	70.3	106	96.1	144	93.8	141
43	27.0	40.4	3.78	5.66	39.3	58.8	61.6	92.4	77.4	116	83.3	125
38	27.0	40.6	3.95	5.93	39.8	59.9	57.1	85.9	78.8	118	87.4	131
34	22.3	33.4	3.51	5.26	33.7	50.5	51.2	77.0	66.5	99.8	79.7	120
30	18.5	27.8	3.56	5.34	30.1	45.2	47.0	70.4	59.4	88.8	74.7	112
26	18.2	27.3	2.73	4.10	27.1	40.6	45.0	67.7	53.5	80.2	70.9	106
22	13.6	20.4	2.55	3.82	21.9	32.8	39.0	58.5	43.2	64.8	63.2	94.8
336	888	1330	108	163	—	—	—	—	1240	1860	597	896
305	744	1120	93.6	140	—	—	—	—	1070	1610	530	796
279	646	970	88.1	132	557	836	557	836	948	1420	485	728
252	537	806	75.4	113	482	724	482	724	813	1220	430	645
230	455	683	65.6	98.4	425	638	425	638	711	1070	387	580
210	384	576	56.6	84.9	374	560	374	560	620	928	347	521
190	313	470	46.0	69.0	321	481	321	481	527	790	305	457
170	256	383	38.9	58.4	277	415	277	415	450	674	269	404
152	209	313	33.0	49.6	239	358	239	358	384	575	239	358
136	170	255	28.3	42.5	207	310	207	310	329	492	212	318
120	136	204	23.8	35.7	178	266	178	266	279	417	186	279
106	103	155	17.1	25.7	147	220	147	220	227	341	157	236
96	84.3	126	14.0	21.0	128	192	128	192	197	295	140	210
87	72.0	108	13.0	19.4	114	171	116	174	177	265	129	194
79	59.7	89.6	11.0	16.5	95.4	143	103	155	155	233	116	175
72	49.9	74.8	9.33	14.0	80.2	120	91.9	138	137	206	105	158
65	40.9	61.4	7.79	11.7	66.2	99.4	81.3	122	120	181	94.5	142
58	38.1	57.2	5.76	8.64	56.8	85.3	76.2	114	111	167	87.8	132
53	33.6	50.3	5.70	8.56	52.1	78.1	71.2	107	102	153	83.2	125
50	39.5	59.3	6.25	9.38	59.8	89.8	75.1	113	110	166	90.2	135
45	32.3	48.4	5.22	7.83	49.3	73.8	66.4	99.6	96.3	144	80.8	121
40	25.3	37.9	4.02	6.03	38.4	57.5	56.9	85.5	75.0	112	70.4	106
35	26.0	39.1	4.00	6.00	39.0	58.6	53.0	79.5	73.5	110	75.0	113
30	19.3	28.9	3.12	4.68	29.4	44.1	44.2	66.4	57.6	86.4	64.2	96.3
26	14.8	22.3	2.52	3.79	23.0	34.6	37.9	57.0	45.1	67.7	56.2	84.3

– Indicates that 3.25 in. bearing length is insufficient for end beam reactions since $N < k$.
N = length of bearing.
x = location of concentrated force with respect to the member end.

F_y = 50 ksi

Table 9–4 (continued)
Beam Bearing Constants

Shape	R_1*		R_2		R_3**		R_4**	
	kips		kips/in.		kips		kips/in.	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	R_1/Ω	ϕR_1	R_2/Ω	ϕR_2	R_3/Ω	ϕR_3	R_4/Ω	ϕR_4
W12×22	15.7	23.6	8.67	13.0	20.8	31.2	2.43	3.64
×19	12.7	19.1	7.83	11.8	16.2	24.3	2.20	3.30
×16	10.4	15.5	7.33	11.0	12.8	19.2	2.42	3.63
×14	8.75	13.1	6.67	10.0	10.2	15.3	2.16	3.24
W10×112	110	165	25.2	37.8	177	265	21.9	32.8
×100	91.8	138	22.7	34.0	143	214	18.3	27.4
×88	75.1	113	20.2	30.3	113	169	14.9	22.4
×77	60.5	90.8	17.7	26.5	86.7	130	11.7	17.5
×68	49.7	74.6	15.7	23.5	68.1	102	9.37	14.1
×60	41.3	61.9	14.0	21.0	54.1	81.1	7.70	11.6
×54	34.4	51.6	12.3	18.5	42.5	63.8	5.90	8.85
×49	30.0	45.1	11.3	17.0	35.7	53.6	5.07	7.61
W10×45	32.7	49.0	11.7	17.5	39.3	58.9	4.95	7.42
×39	27.0	40.6	10.5	15.8	31.0	46.5	4.30	6.44
×33	22.6	33.9	9.67	14.5	24.8	37.2	4.16	6.24
W10×30	20.3	30.4	10.0	15.0	28.3	42.4	3.65	5.48
×26	16.0	24.1	8.67	13.0	21.2	31.8	2.79	4.19
×22	13.2	19.8	8.00	12.0	17.0	25.5	2.73	4.09
W10×19	14.5	21.7	8.33	12.5	18.9	28.4	2.79	4.19
×17	12.6	18.9	8.00	12.0	16.3	24.4	2.99	4.49
×15	10.9	16.4	7.67	11.5	13.8	20.7	3.26	4.88
×12	8.07	12.1	6.33	9.50	9.14	13.7	2.39	3.59
W8×67	63.1	94.7	19.0	28.5	100	150	15.9	23.9
×58	51.2	76.8	17.0	25.5	78.9	118	13.5	20.3
×48	36.0	54.0	13.3	20.0	50.4	75.6	7.94	11.9
×40	28.6	42.9	12.0	18.0	38.9	58.4	7.30	10.9
×35	23.0	34.4	10.3	15.5	29.2	43.9	5.35	8.03
×31	19.7	29.5	9.50	14.3	24.2	36.3	4.81	7.21
W8×28	20.4	30.6	9.50	14.3	25.0	37.5	4.46	6.69
×24	16.2	24.3	8.17	12.3	18.5	27.7	3.35	5.02
W8×21	14.6	21.9	8.33	12.5	19.0	28.6	3.41	5.11
×18	12.1	18.1	7.67	11.5	15.3	22.9	3.27	4.91
W8×15	12.6	18.8	8.17	12.3	16.4	24.6	4.16	6.24
×13	10.6	16.0	7.67	11.5	13.4	20.1	4.31	6.47
×10	7.15	10.7	5.67	8.50	7.64	11.5	2.19	3.29

* When compressive force is applied at a distance greater than *d* from the beam end, this value may be multiplied by two.

** When compressive force is applied at a distance greater than *d*/2 from the beam end, this value may be multiplied by two.

Table 9–4 (continued)
Beam Bearing Constants

$F_y = 50$ ksi

Nominal Wt.	R_5		R_6		($N = 3^1/_4$)						V_{nx}/Ω_v	$\phi_v V_{nx}$
					$x < d/2$		$d/2 \le x \le d$		$x > d$			
	kips		kips/in.		kips		kips		kips			
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	kips	kips
lb/ft	R_5/Ω	ϕR_5	R_6/Ω	ϕR_6	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	R_n/Ω	ϕR_n	ASD	LRFD
22	18.8	28.2	3.24	4.85	29.3	44.0	43.9	65.9	57.4	86.1	64.0	96.0
19	14.4	21.7	2.94	4.41	24.0	36.0	38.1	57.5	46.7	70.1	57.2	85.7
16	10.9	16.3	3.23	4.84	21.4	32.0	34.2	51.3	41.3	62.0	52.8	79.1
14	8.51	12.8	2.88	4.31	17.9	26.8	30.4	45.6	34.4	51.7	42.8	64.3
112	160	240	29.2	43.8	192	288	192	288	302	453	172	257
100	129	194	24.4	36.5	166	249	166	249	257	387	151	226
88	102	153	19.9	29.8	141	211	141	211	216	324	131	197
77	78.4	118	15.6	23.3	118	177	118	177	179	268	112	169
68	61.6	92.4	12.5	18.7	101	151	101	151	150	226	97.8	147
60	48.8	73.2	10.3	15.4	82.3	123	86.8	130	128	192	85.8	129
54	38.5	57.8	7.86	11.8	64.0	96.1	74.4	112	109	163	74.7	112
49	32.3	48.5	6.76	10.1	54.3	81.3	66.7	100	96.7	145	68.0	102
45	35.9	53.9	6.60	9.89	57.4	86.0	70.7	106	103	155	70.7	106
39	28.2	42.2	5.73	8.59	46.8	70.1	61.1	92.0	88.1	133	62.5	93.7
33	22.1	33.2	5.55	8.33	40.1	60.3	54.0	81.0	76.6	115	56.4	84.7
30	25.7	38.6	4.87	7.31	41.5	62.4	52.8	79.2	73.1	110	62.8	94.2
26	19.3	28.9	3.73	5.59	31.4	47.1	44.2	66.4	60.2	90.4	53.7	80.6
22	15.1	22.7	3.64	5.46	26.9	40.4	39.2	58.8	51.7	77.6	48.8	73.2
19	17.0	25.5	3.72	5.58	29.1	43.6	41.6	62.3	55.9	84.0	51.2	76.8
17	14.2	21.4	3.99	5.99	27.2	40.9	38.6	57.9	51.2	76.8	48.5	72.8
15	11.6	17.4	4.34	6.51	25.7	38.6	35.8	53.8	46.7	70.2	46.0	69.0
12	7.57	11.4	3.19	4.78	17.9	26.9	28.6	43.0	33.8	50.7	37.5	56.3
67	90.7	136	21.2	31.8	125	187	125	187	188	282	103	154
58	71.1	107	18.0	27.0	106	160	106	160	158	236	89.3	134
48	45.9	68.9	10.6	15.9	79.2	119	79.2	119	115	173	68.0	102
40	34.9	52.4	9.73	14.6	66.5	99.9	67.6	101	96.2	144	59.4	89.1
35	26.3	39.5	7.14	10.7	49.5	74.3	56.5	84.8	79.5	119	50.3	75.5
31	21.6	32.4	6.41	9.61	42.4	63.6	50.6	76.0	70.3	105	45.6	68.4
28	22.6	33.9	5.95	8.93	41.9	62.9	51.3	77.1	71.7	108	45.9	68.9
24	16.7	25.1	4.47	6.70	31.2	46.9	42.8	64.3	58.8	88.0	38.9	58.3
21	17.2	25.7	4.54	6.82	32.0	47.9	41.7	62.5	56.3	84.4	41.4	62.1
18	13.5	20.2	4.36	6.55	27.7	41.5	37.0	55.5	49.1	73.6	37.4	56.2
15	14.1	21.2	5.55	8.32	32.1	48.2	39.2	58.8	51.8	77.6	39.7	59.6
13	11.1	16.7	5.75	8.63	29.8	44.7	35.5	53.4	46.1	69.4	36.8	55.1
10	6.49	9.73	2.93	4.39	16.0	24.0	25.6	38.3	29.5	44.4	26.8	40.2

N = length of bearing.
x = location of concentrated force with respect to the member end.

PART 10

DESIGN OF SIMPLE SHEAR CONNECTIONS

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of simple shear connections. For the design of flexible moment connections, see Part 11. For the design of fully restrained (FR) moment connections, see Part 12.

FORCE TRANSFER

The required strength (end reaction), R_u or R_a, is determined by analysis as indicated in AISC Specification Section B3. Per AISC Specification Section J1.2, the ends of members with simple shear connections are normally assumed to be free to rotate under load. While simple shear connections do actually possess some rotational restraint (see curve A in Figure 10–1), this small amount can be neglected and the connection idealized as completely flexible. The simple shear connections shown in this Manual are suitable to accommodate the end rotations required per AISC Specification Section J1.2.

Support rotation is acceptably limited for most framing details involving simple shear connections without explicit consideration. The case of a bare spandrel girder supporting infill beams, however, may require consideration to verify that an acceptable level of support rotational stiffness is present. Sumner (2003) showed that a nominal interconnection between the top flange of the girder and the top flange of the framing beam is sufficient to limit support rotation.

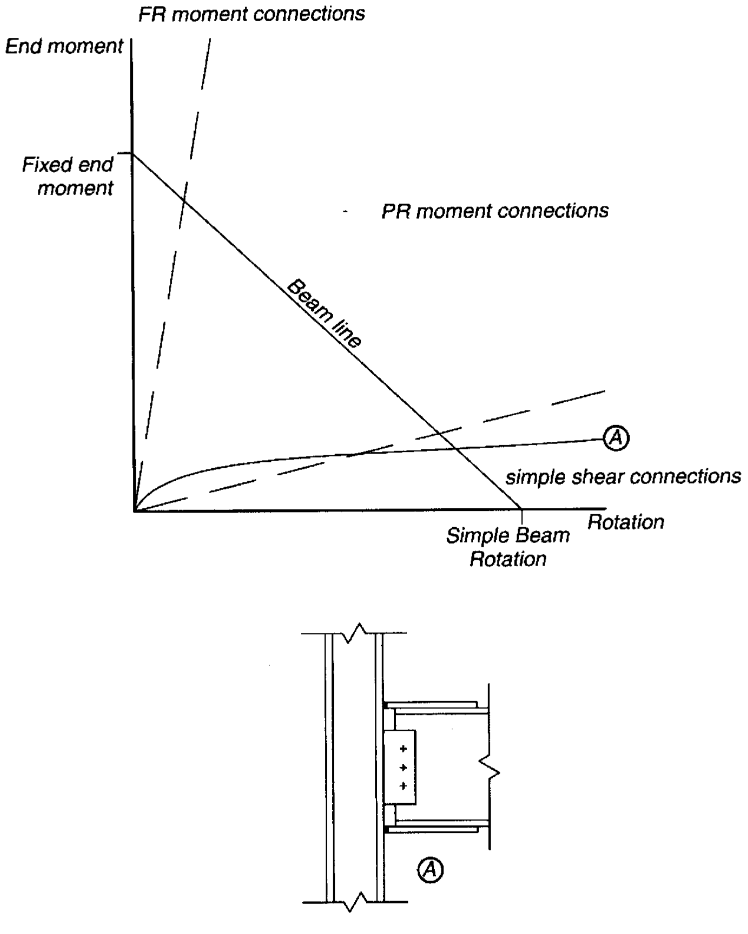


Figure 10–1. Illustration of typical moment rotation curve for simple shear connection.

COMPARING CONNECTION ALTERNATIVES

Two-Sided Connections

Two-sided connections, such as double-angle and shear end-plate connections, offer the following advantages:

1. suitability for use when the end reaction is large;
2. compact connections (usually, the entire connection is contained within the flanges of the supported beam); and,
3. eccentricity perpendicular to the beam axis need not be considered for workable gages (see Table 1–7).

Note that two-sided connections may require additional consideration for erectability, as discussed in "Constructability Considerations" below.

Seated Connections

Unstiffened and stiffened seated connections offer the following advantages:

1. seats can be shop attached to the support, simplifying erection;
2. ample erection clearance is provided;
3. excellent safety during erection since double connections often can be eliminated; and,
4. the bay length of the structure is easily maintained (seated connections may be preferable when maintaining bay length is a concern for repetitive bays of framing).

Note that seated connections can cause erection interference when floors are close, beams are deep, or seats protrude excessively from the column face. The practice of leaning or tilting the columns to erect a column-web connection is difficult, unsafe and should always be avoided.

One-Sided Connections

One-sided connections such as single-plate, single-angle and tee connections offer the following advantages:

1. shop attachment of connection elements to the support, simplifying shop fabrication and erection;
2. reduced material and shop labor requirements;
3. ample erection clearance is provided; and,
4. excellent safety during erection since double connections often can be eliminated.

CONSTRUCTABILITY CONSIDERATIONS

Double Connections

A double connection occurs in field-bolted construction when beams or girders frame opposite each other. Double connections are of concern to OSHA when they occur in the web of a column (see Figure 10–2) or the web of a beam that frames continuously over the top of a column[1] and all field bolts take the same open holes. A positive connection must be

---

[1] This requirement applies only at the location of the column, not at locations away from the column.

made and maintained for the first member to be erected while the second member to be erected is brought into its final position. Conditions requiring the connector to hang one beam temporarily on a partially inserted bolt or drift pin are not allowed by OSHA.

Framing details can be configured using staggered angles or other similar details to provide a means to make a positive connection for the first member while the second member is brought into its final position. Alternatively, a temporary erection seat, as shown in Figure 10–2, can be provided. The erection seat, usually an angle, is sized and attached to the column web to support the dead weight of the member, unless additional loading is indicated in the contract documents. It is located to clear the bottom flange of the supported member by approximately $^3/_8$ in. to accommodate mill, fabrication, and erection tolerances.

The sequence of erection is most important in determining the need for erection seats. If the erection sequence is known, the erection seat is provided on the side needing the support. If the erection sequence is not known, a seat can be provided on both sides of the column web. Temporary erection seats may be reused at other locations after the connection(s) are made, but need not be removed unless they create an interference or removal is required in the contract documents.

See also the discussion under "Special Considerations for Simple Shear Connections."

Accessibility in Column Webs

Because of bolting and welding clearances, double-angle, shear end-plate, single-plate, single-angle, and tee shear connections may not be suitable for connections to the webs of W-shape and similar columns, particularly for W8 columns, unless gages are reduced. Such connections may be impossible for W6, W5, and W4 columns.

There is also an accessibility concern for entering and tightening the field bolts when the connection material is shop-attached to the supporting column web and contained within the column flanges.

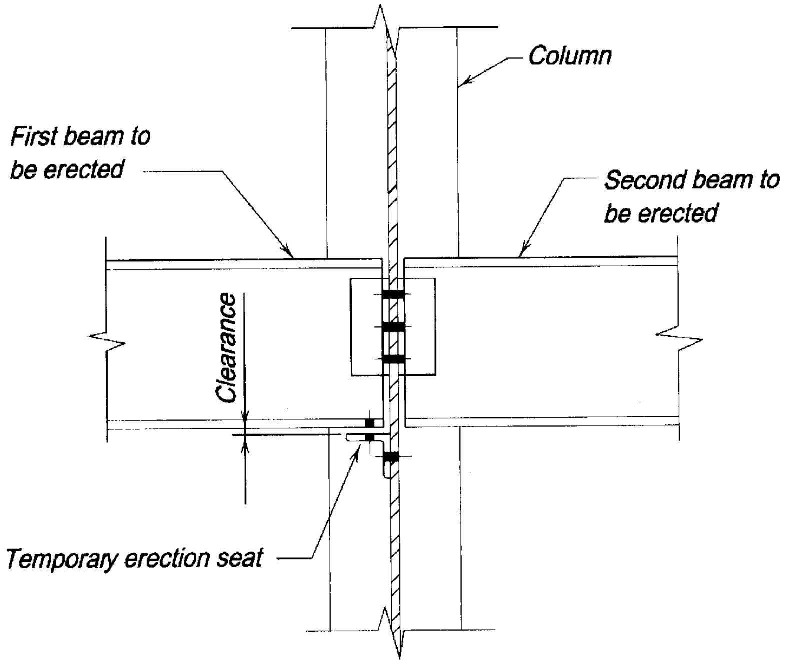


Figure 10–2. Erection seat.

Field-Welded Connections

In field-welded connections, temporary erection bolts are usually provided to support the member until final welding is performed. A minimum of 2 bolts (one bolt in bracing members) must be placed for erection safety per OSHA requirements. Additional erection bolts may be required for loads during erection, to assist in pulling the connection angles up tightly against the web of the supporting beam prior to welding or for other reasons. Temporary erection bolts may be reused at other locations after final welding, but need not be removed unless they create an interference or removal is required in the contract documents.

Riding the Fillet

The detailed dimensions of connection elements must be compatible with the T-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam. Note that the angle may encroach upon the fillet(s), as given in Figure 10–3.

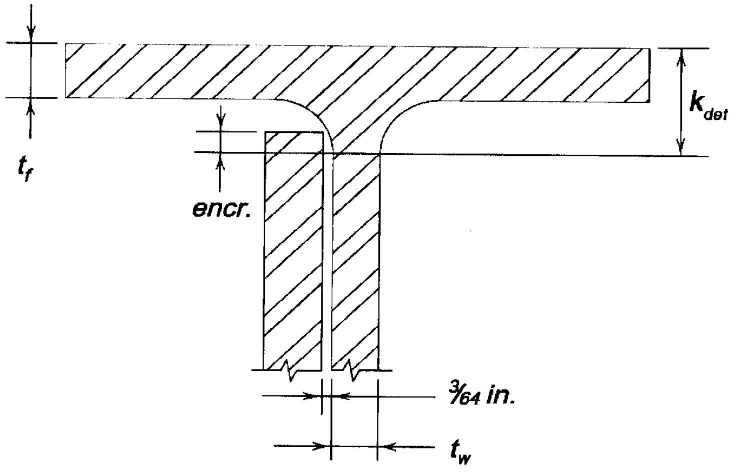
DOUBLE-ANGLE CONNECTIONS

A double-angle connection is made with two angles, one on each side of the web of the beam to be supported, as illustrated in Figure 10–4. These angles may be bolted or welded to the supported beam as well as to the supporting member.

When the angles are welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10–4c, line welds are placed along the toes of the angles with a return at the top per AISC Specification Section J2.2b. Note that welding across the entire top of the angles must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for simple shear connections.

Available Strength

The available strength of a double-angle connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.



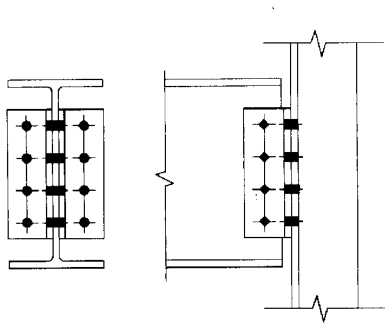
$k_{det} - t_f$	Encr.
in.	in.
$^5/_{16}$	$^1/_8$
$^3/_8$ to $^1/_2$	$^3/_{16}$
$^9/_{16}$ to $^{13}/_{16}$	$^1/_4$
$^7/_8$ to 1 $^1/_4$	$^5/_{16}$
1 $^5/_{16}$ and 1 $^3/_8$	$^3/_8$

Figure 10–3. Fillet encroachment (riding the fillet).

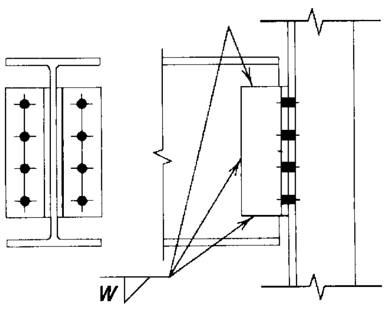
For the workable gages shown in Table 1–7 and standard or short-slotted holes, eccentricity in double-angle connections may be neglected, except in the case of a double vertical row of bolts through the web of the supported beam. Eccentricity should always be considered in the design of welds for double-angle connections.

Recommended Angle Length and Thickness

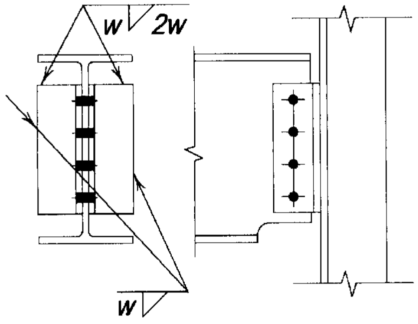
To provide for stability during erection, it is recommended that the minimum angle length be one-half the *T*-dimension of the beam to be supported. The maximum length of the connection angles must be compatible with the *T*-dimension of an uncoped beam and the



(a) All-bolted



(b) Bolted/welded, angles welded to supported beam



Note: weld returns on top of angles per Specification Section J2.2b.

(c) Bolted/welded, angles welded to support

Figure 10–4. Double-angle connections.

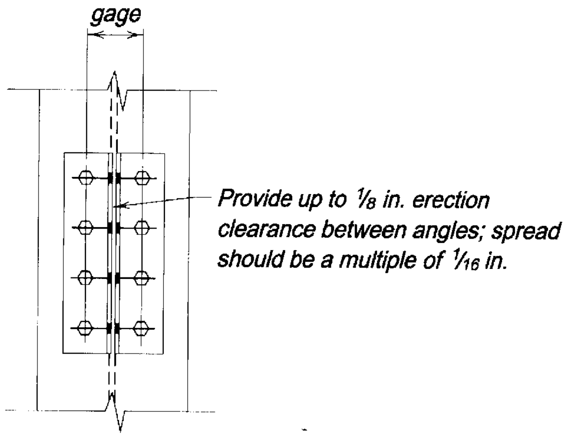
remaining web depth, exclusive of fillets, of a coped beam. Note that the angle may encroach upon the fillet(s), as given in Figure 10–3.

To provide for flexibility, the maximum angle thickness for use with workable gages should be limited to $^5/_8$ in. Alternatively, the shear-connection ductility checks illustrated in Part 9 can be used to justify other combinations of gage and angle thickness.

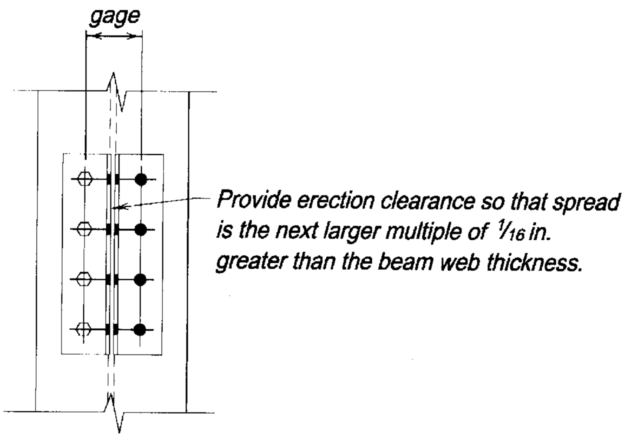
Shop and Field Practices

When framing to a girder web, both angles are usually shop-attached to the web of the supported beam. When framing to a column web, both angles should be shop-attached to the supported beam, when possible, and the associated constructability considerations should be addressed (see the preceding discussion under "Constructability Considerations").

When framing to a column flange, both angles can be shop-attached to the column flange or the supported beam. In the former case, this is a knifed connection, as illustrated in Figure 10–4c, which requires an erection clearance, as illustrated in Figure 10–5a, and that the bottom flange be coped away. Also, provision must be made for possible mill variation in



(a) Both angles shop attached to the column flange (beam knifed into place)



(b) One shop attached to the column flange, other shipped loose

Figure 10–5. Erection clearances for double-angle connections.

the depth of the columns, particularly in fairly long runs (i.e., six or more bays of framing). If both angles are shop-attached to the beam web, the beam length can be shortened to provide for mill overrun with shims furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun. If both angles are shop-attached to the column flange, the erected beam is knifed into place and play in the open holes is normally sufficient to provide for the necessary adjustment. Alternatively, short-slotted holes can also be used.

When special requirements preclude the use of any of the foregoing practices, one angle could be shop-attached to the support and the other shipped loose. In this case, the spread between the outstanding legs should equal the decimal beam web thickness plus a clearance that will produce an opening to the next higher $^1/_{16}$-in. increment, as illustrated in Figure 10–5b. Alternatively, short-slotted holes in the support leg of the angle eliminate the need to provide for variations in web thickness. Note that the practice of shipping one angle loose is not desirable because it requires additional material handling as well as added erection costs and difficulty.

Table 10–1. All-Bolted Double-Angle Connections

Table 10–1 is a design aid for all-bolted double-angle connections. Available strengths are tabulated for supported and supporting member material with $F_y = 50$ ksi and $F_u = 65$ ksi and angle material with $F_y = 36$ ksi and $F_u = 58$ ksi. All values, including slip-critical bolt available strengths, are for comparison with the LRFD load combination for LRFD design and the ASD load combination for ASD design.

Tabulated bolt and angle available strengths consider the limit states of bolt shear, bolt bearing on the angles, shear yielding of the angles, shear rupture of the angles, and block shear rupture of the angles. Values are tabulated for 2 through 12 rows of $^3/_4$-in., $^7/_8$-in, and 1-in. diameter ASTM A325, F1852, and A490 bolts at 3-in. spacing. For calculation purposes, angle edge distances, L_{ev} and L_{eh}, are assumed to be $1^1/_4$ in.

Tabulated beam web available strengths, per in. of web thickness, consider the limit-state of bolt bearing on the beam web. For beams coped at the top flange only, the limit-state of block shear rupture is also considered. Additionally, for beams coped at both the top and bottom flanges, the tabulated values consider the limit-states of shear yielding and shear rupture of the beam web. Values are tabulated for beam web edge distances L_{ev} from $1^1/_4$ in. to 3 in. and for beam end distances, L_{eh}, of $1^1/_2$ in. and $1^3/_4$ in. For calculation purposes, these end distances have been reduced to $1^1/_4$ in. and $1^1/_2$ in., respectively, to account for possible underrun in beam length. For coped members, the limit states of flexural yielding and local buckling must be checked independently per Part 9. When required, web reinforcement of coped members is treated as in Part 9.

Tabulated supporting member available strengths, per in. of flange or web thickness, consider the limit-state of bolt bearing on the support. Note that resistance and safety factors are not noted in these tables, as they vary by limit state.

Table 10–2. Bolted/Welded Double-Angle Connections

Tables 10–2 is a design aid arranged to permit substitution of welds for bolts in connections designed with Tables 10–1. Electrode strength is assumed to be 70 ksi. Holes for erection bolts may be placed as required in angle legs that are to be field-welded.

Welds A may be used in place of bolts through the supported-beam web legs of the double angles or welds B may be used in place of bolts through the support legs of the double angles. Although it is permissible to use welds A and B from Table 10–2 in combination to obtain all-welded connections, it is recommended that such connections be selected from Table 10–3. This table will allow increased flexibility in the selection of angle lengths and connection strengths because Table 10–2 conforms to the bolt spacing and edge distance requirements for the all-bolted double-angle connections of Table 10–1.

Weld available strengths are tabulated for the limit-state of weld shear. Available strengths for welds A are determined by the instantaneous center of rotation method using Table 8–8 with $\theta = 0°$. Available strengths for welds B are determined by the elastic method. With the neutral axis assumed at one-sixth the depth of the angles measured downward and the tops of the angles in compression against each other through the beam web, the available strength, ϕR_n or R_n/Ω, of these welds is determined by

LRFD	ASD
$\phi R_n = 2\times\dfrac{1.392DL}{\sqrt{1+\dfrac{12.96e^2}{L^2}}}$	$\dfrac{R_n}{\Omega} = 2\times\dfrac{0.928DL}{\sqrt{1+\dfrac{12.96e^2}{L^2}}}$

where

D = number of sixteenths-of-an-inch in the weld size.
L = length of the connection angles, in.
e = width of the leg of the connection angle attached to the support, in.

The tabulated minimum thicknesses of the supported beam web for welds A and the support for welds B match the shear rupture strength of these elements with the strength of the weld metal. As derived in Part 9, the minimum supported beam web thickness for welds A (two lines of weld) is

$$t_{min} = \frac{6.19D}{F_u}$$

and the minimum supporting flange or web thickness welds B (one line of weld) is

$$t_{min} = \frac{3.09D}{F_u}$$

When welds B line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

When Table 10–2 is used, the minimum angle thickness is the weld size plus $^1/_{16}$ in., but not less than the angle thickness determined from Table 10–1. The angle length L must be as tabulated in Table 10–2. In general, 2L4×3$^1/_2$ will accommodate workable gages, with the 4-in. leg attached to the supporting member. The width of web legs in Case I may be optionally reduced from 3$^1/_2$ in. to 3 in. The width of outstanding legs in Case II may be optionally reduced from 4 in. to 3 in. for values of L from 5$^1/_2$ through 17$^1/_2$ in.

Table 10–3. All-Welded Double-Angle Connections

Table 10–3 is a design aid for all-welded double-angle connections. Electrode strength is assumed to be 70 ksi. Holes for erection bolts may be placed as required in angle legs that are to be field-welded.

Weld available strengths are tabulated for the limit-state of weld shear. Available strengths for welds A are determined by the instantaneous center of rotation method using Table 8–8 with $\theta = 0°$. Available strengths for welds B are determined by the elastic method as discussed previously for bolted/welded double-angle connections.

The tabulated minimum thicknesses of the supported beam web for welds A and the support for welds B match the shear rupture strength of these elements with the strength of the weld metal and are determined as discussed previously for Table 10–2. When welds B line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. When less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

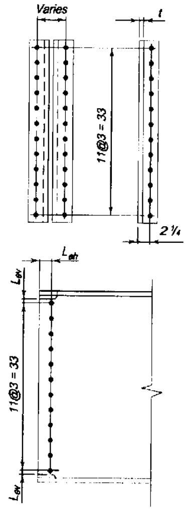
When Table 10–3 is used, the minimum angle thickness must be equal to the weld size plus $^1/_{16}$ in. The angle length, *L*, must be as tabulated in Table 10–3. 2L4×3 should be used for angle lengths equal to or greater than 18 in. 2L3×3 should be used otherwise.

Beam	F_y = 50 ksi F_u = 65 ksi	
Angle	F_y = 36 ksi F_u = 58 ksi	

Table 10–1
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

12 Rows W44	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD	$^1/_2$ ASD	$^1/_2$ LRFD
	A325/ F1852	N	—	197	295	246	369	254	382	254	382
		X	—	197	295	246	369	295	443	318	477
		SC Class A	STD	177	266	177	266	177	266	177	266
			OVS	128	192	128	192	128	192	128	192
			SSLT	151	226	151	226	151	226	151	226
		SC Class B	STD	197	295	246	369	253	380	253	380
			OVS	183	274	183	274	183	274	183	274
			SSLT	195	293	215	323	215	323	215	323
	A490	N	—	197	295	246	369	295	443	318	477
		X	—	197	295	246	369	295	443	393	590
		SC Class A	STD	197	295	221	332	221	332	221	332
			OVS	160	240	160	240	160	240	160	240
			SSLT	188	282	188	282	188	282	188	282
		SC Class B	STD	197	295	246	369	295	443	316	475
			OVS	196	294	229	343	229	343	229	343
			SSLT	195	293	244	366	269	403	269	403

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		$1^1/_2$		$1^3/_4$		$1^1/_2$		$1^3/_4$		$1^1/_2$		$1^3/_4$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	$1^1/_4$	498	747	506	759	468	702	476	714	495	743	503	755
	$1^3/_8$	500	751	509	763	470	706	479	718	497	746	506	758
	$1^1/_2$	503	754	511	767	473	709	481	722	500	750	508	762
	$1^5/_8$	505	758	514	770	475	713	483	725	502	753	510	766
	2	513	769	521	781	483	724	491	736	510	764	518	777
	3	532	798	540	810	502	753	510	765	529	794	537	806
Coped at Both Flanges	$1^1/_4$	488	731	488	731	458	687	458	687	488	731	488	731
	$1^3/_8$	492	739	492	739	463	695	463	695	492	739	492	739
	$1^1/_2$	497	746	497	746	468	702	468	702	497	746	497	746
	$1^5/_8$	502	753	502	753	473	709	473	709	502	753	502	753
	2	513	769	517	775	483	724	488	731	510	764	517	775
	3	532	798	540	810	502	753	510	765	529	794	537	806
Uncoped		702	1050	702	1050	702	1050	702	1050	702	1050	702	1050

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	**1400**	2110

Notes:

STD = Standard holes

OVS = Oversized holes

SSLT = Short-slotted holes transverse to direction of load

N = Threads included

X = Threads excluded

SC = Slip critical

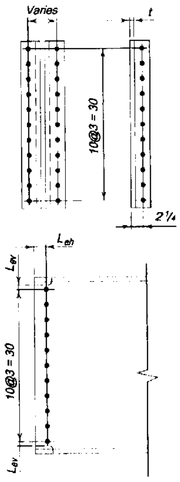
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

11 Rows W44, 40	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness ¼ ASD	¼ LRFD	5/16 ASD	5/16 LRFD	⅜ ASD	⅜ LRFD	½ ASD	½ LRFD
	A325/ F1852	N	—	181	271	226	338	233	350	233	350
		X	—	181	271	226	338	271	406	292	437
		SC Class A	STD	162	244	162	244	162	244	162	244
			OVS	117	176	117	176	117	176	117	176
			SSLT	138	207	138	207	138	207	138	207
		SC Class B	STD	181	271	226	338	232	348	232	348
			OVS	168	251	168	251	168	251	168	251
			SSLT	179	269	197	296	197	296	197	296
	A490	N	—	181	271	226	338	271	406	292	437
		X	—	181	271	226	338	271	406	361	542
		SC Class A	STD	181	271	203	305	203	305	203	305
			OVS	147	220	147	220	147	220	147	220
			SSLT	173	259	173	259	173	259	173	259
		SC Class B	STD	181	271	226	338	271	406	290	435
			OVS	80	269	210	314	210	314	210	314
			SSLT	179	269	224	336	247	370	247	370

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
	L_{eh}*	1½		1¾		1½		1¾		1½		1¾	
L_{ev}, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	457	685	465	697	429	644	437	656	454	680	462	693
	1⅜	459	689	467	701	431	647	440	659	456	684	464	696
	1½	462	692	470	704	434	651	442	663	458	688	467	700
	1⅝	464	696	472	708	436	654	444	667	461	691	469	704
	2	471	707	479	719	444	665	452	678	468	702	476	714
	3	491	736	499	748	463	695	471	707	488	732	496	744
Coped at Both Flanges	1¼	446	669	446	669	419	629	419	629	446	669	446	669
	1⅜	451	676	451	676	424	636	424	636	451	676	451	676
	1½	456	684	456	684	429	644	429	644	456	684	456	684
	1⅝	461	691	461	691	434	651	434	651	461	691	461	691
	2	471	707	475	713	444	665	449	673	468	702	475	713
	3	491	736	499	748	463	695	471	707	488	732	496	744
Uncoped		644	965	644	965	644	965	644	965	644	965	644	965

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1290	1930

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

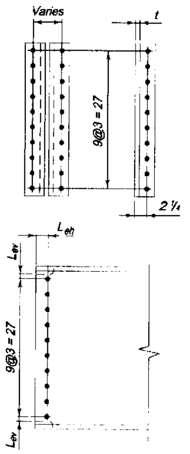
* Tabulated values include ¼-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi F_u = 65 ksi	F_y = 36 ksi F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

10 Rows W44, 40, 36	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	164	246	205	308	212	318	212	318
		X	—	164	246	205	308	246	370	265	398
		SC Class A	STD	148	221	148	221	148	221	148	221
			OVS	107	160	107	160	107	160	107	160
			SSLT	126	188	126	188	126	188	126	188
		SC Class B	STD	164	246	205	308	211	316	211	316
			OVS	152	229	152	229	152	229	152	229
			SSLT	163	244	179	269	179	269	179	269
	A490	N	—	164	246	205	308	246	370	265	398
		X	—	164	246	205	308	246	370	329	493
		SC Class A	STD	164	246	185	277	185	277	185	277
			OVS	133	200	133	200	133	200	133	200
			SSLT	157	235	157	235	157	235	157	235
		SC Class B	STD	164	246	205	308	246	370	264	396
			OVS	163	245	190	286	190	286	190	286
			SSLT	163	244	204	306	224	336	224	336

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	415	623	423	635	390	585	398	597	412	618	420	630
	1 3/8	418	626	426	639	392	589	401	601	415	622	423	634
	1 1/2	420	630	428	642	395	592	403	605	417	626	425	638
	1 5/8	423	634	431	646	397	596	405	608	419	629	428	641
	2	430	645	438	657	405	607	413	619	427	640	435	652
	3	449	674	457	686	424	636	432	648	446	669	454	682
Coped at Both Flanges	1 1/4	405	607	405	607	380	570	380	570	405	607	405	607
	1 3/8	410	614	410	614	385	578	385	578	410	614	410	614
	1 1/2	414	622	414	622	390	585	390	585	414	622	414	622
	1 5/8	419	629	419	629	395	592	395	592	419	629	419	629
	2	430	645	434	651	405	607	410	614	427	640	434	651
	3	449	674	457	686	424	636	432	648	446	669	454	682
Uncoped		585	878	585	878	585	878	585	878	585	878	585	878

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1170	1760

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

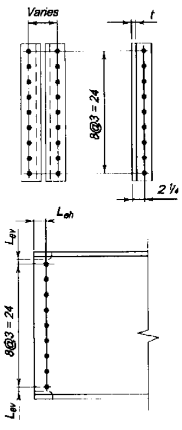
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

9 Rows W44, 40, 36, 33	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	148	222	185	278	191	286	191	286
		X	—	148	222	185	278	222	333	239	358
		SC Class A	STD	133	199	133	199	133	199	133	199
			OVS	96	144	96	144	96	144	96	144
			SSLT	113	169	113	169	113	169	113	169
		SC Class B	STD	148	222	185	278	190	285	190	285
			OVS	137	206	137	206	137	206	137	206
			SSLT	147	220	161	242	161	242	161	242
	A490	N	—	148	222	185	278	222	333	239	358
		X	—	148	222	185	278	222	333	296	444
		SC Class A	STD	148	222	166	249	166	249	166	249
			OVS	120	180	120	180	120	180	120	180
			SSLT	141	212	141	212	141	212	141	212
		SC Class B	STD	148	222	185	278	222	333	237	356
			OVS	147	221	171	257	171	257	171	257
			SSLT	147	220	183	275	202	303	202	303

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	374	561	382	573	351	527	359	539	371	556	379	568
	1 3/8	376	564	384	576	353	530	362	542	373	560	381	572
	1 1/2	379	568	387	580	356	534	364	546	376	563	384	576
	1 5/8	381	572	389	584	358	537	366	550	378	567	386	579
	2	388	583	397	595	366	548	374	561	385	578	393	590
	3	408	612	416	624	385	578	393	590	405	607	413	619
Coped at Both Flanges	1 1/4	363	545	363	545	341	512	341	512	363	545	363	545
	1 3/8	368	552	368	552	346	519	346	519	368	552	368	552
	1 1/2	373	559	373	559	351	527	351	527	373	559	373	559
	1 5/8	378	567	378	567	356	534	356	534	378	567	378	567
	2	388	583	392	589	366	548	371	556	385	578	392	589
	3	408	612	416	624	385	578	393	590	405	607	413	619
Uncoped		526	790	526	790	526	790	526	790	526	790	526	790

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1050	1580

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

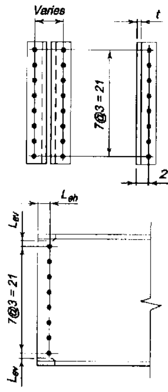
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

8 Rows	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
W44, 40, 36, 33, 30				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	132	198	165	247	170	254	170	254
		X	—	132	198	165	247	198	297	212	318
		SC Class A	STD	118	177	118	177	118	177	118	177
			OVS	85.3	128	85.3	128	85.3	128	85.3	128
			SSLT	100	151	100	151	100	151	100	151
		SC Class B	STD	132	198	165	247	169	253	169	253
			OVS	122	183	122	183	122	183	122	183
			SSLT	131	196	143	215	143	215	143	215
	A490	N	—	132	198	165	247	198	297	212	318
		X	—	132	198	165	247	198	297	264	396
		SC Class A	STD	132	198	148	221	148	221	148	221
			OVS	107	160	107	160	107	160	107	160
			SSLT	126	188	126	188	126	188	126	188
		SC Class B	STD	132	198	165	247	198	297	211	316
			OVS	131	197	152	229	152	229	152	229
			SSLT	131	196	163	245	179	269	179	269

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	332	498	340	511	312	468	320	480	329	494	337	506
	1 3/8	335	502	343	514	314	472	323	484	332	498	340	510
	1 1/2	337	506	345	518	317	475	325	488	334	501	342	513
	1 5/8	340	509	348	522	319	479	327	491	337	505	345	517
	2	347	520	355	533	327	490	335	502	344	516	352	528
	3	366	550	375	562	346	519	354	531	363	545	372	557
Coped at Both Flanges	1 1/4	322	483	322	483	302	453	302	453	322	483	322	483
	1 3/8	327	490	327	490	307	461	307	461	327	490	327	490
	1 1/2	332	497	332	497	312	468	312	468	332	497	332	497
	1 5/8	336	505	336	505	317	475	317	475	336	505	336	505
	2	347	520	351	527	327	490	332	497	344	516	351	527
	3	366	550	375	562	346	519	354	531	363	545	372	557
Uncoped		468	702	468	702	468	702	468	702	468	702	468	702

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	936	1400

Notes:

STD = Standard holes

OVS = Oversized holes

SSLT = Short-slotted holes transverse to direction of load

N = Threads included

X = Threads excluded

SC = Slip critical

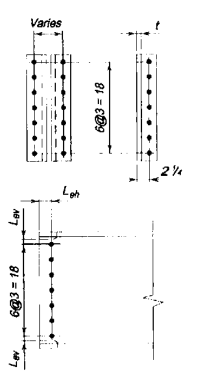
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

7 Rows W44, 40, 36, 33, 30, 27, 24	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	116	174	145	217	148	223	148	223
		X	—	116	174	145	217	174	260	186	278
		SC Class A	STD	103	155	103	155	103	155	103	155
			OVS	74.7	112	74.7	112	74.7	112	74.7	112
			SSLT	87.9	132	87.9	132	87.9	132	87.9	132
		SC Class B	STD	116	174	145	217	148	221	148	221
			OVS	107	160	107	160	107	160	107	160
			SSLT	114	172	126	188	126	188	126	188
	A490	N	—	116	174	145	217	174	260	186	278
		X	—	116	174	145	217	174	260	231	347
		SC Class A	STD	116	174	129	194	129	194	129	194
			OVS	93.3	140	93.3	140	93.3	140	93.3	140
			SSLT	110	165	110	165	110	165	110	165
		SC Class B	STD	116	174	145	217	174	260	185	277
			OVS	115	172	133	200	133	200	133	200
			SSLT	114	172	143	214	157	235	157	235

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	291	436	299	449	273	410	281	422	288	432	296	444
	1 3/8	293	440	301	452	275	413	284	425	290	435	298	448
	1 1/2	296	444	304	456	278	417	286	429	293	439	301	451
	1 5/8	298	447	306	459	280	420	288	433	295	443	303	455
	2	306	458	314	470	288	431	296	444	302	454	311	466
	3	325	488	333	500	307	461	315	473	322	483	330	495
Coped at Both Flanges	1 1/4	280	420	280	420	263	395	263	395	280	420	280	420
	1 3/8	285	428	285	428	268	402	268	402	285	428	285	428
	1 1/2	290	435	290	435	273	410	273	410	290	435	290	435
	1 5/8	295	442	295	442	278	417	278	417	295	442	295	442
	2	306	458	310	464	288	431	293	439	302	454	310	464
	3	325	488	333	500	307	461	315	473	322	483	330	495
Uncoped		410	614	410	614	410	614	410	614	410	614	410	614

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	819	1230

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

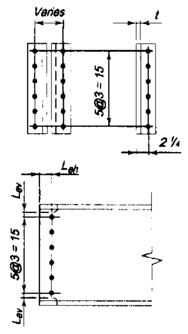
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

6 Rows W40, 36, 33, 30, 27, 24, 21	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	99.5	149	124	187	127	191	127	191
		X	—	99.5	149	124	187	149	224	159	239
		SC Class A	STD	88.6	133	88.6	133	88.6	133	88.6	133
			OVS	64	96	64	96	64	96	64	96
			SSLT	75.3	113	75.3	113	75.3	113	75.3	113
		SC Class B	STD	99.5	149	124	187	127	190	127	190
			OVS	91.4	137	91.4	137	91.4	137	91.4	137
			SSLT	98.2	147	108	161	108	161	108	161
	A490	N	—	99.5	149	124	187	149	224	159	239
		X	—	99.5	149	124	187	149	224	199	298
		SC Class A	STD	99.5	149	111	166	111	166	111	166
			OVS	80	120	80	120	80	120	80	120
			SSLT	94.1	141	94.1	141	94.1	141	94.1	141
		SC Class B	STD	99.5	149	124	187	149	224	158	237
			OVS	98.6	148	114	171	114	171	114	171
			SSLT	98.2	147	123	184	134	202	134	202

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1½		1¾		1½		1¾		1½		1¾	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	249	374	258	386	234	351	242	363	246	370	255	382
	1⅜	252	378	260	390	236	355	245	367	249	373	257	385
	1½	254	381	262	394	239	358	247	371	251	377	259	389
	1⅝	257	385	265	397	241	362	249	374	254	381	262	393
	2	264	396	272	408	249	373	257	385	261	392	269	404
	3	284	425	292	438	268	402	276	414	281	421	289	433
Coped at Both Flanges	1¼	239	358	239	358	224	336	224	336	239	358	239	358
	1⅜	244	366	244	366	229	344	229	344	244	366	244	366
	1½	249	373	249	373	234	351	234	351	249	373	249	373
	1⅝	254	380	254	380	239	358	239	358	254	380	254	380
	2	264	396	268	402	249	373	254	380	261	392	268	402
	3	284	425	292	438	268	402	276	414	281	421	289	433
Uncoped		351	526	351	526	351	526	351	526	351	526	351	526

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	702	1050

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

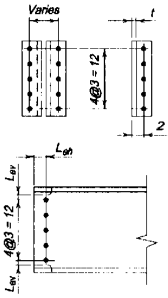
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi F_u = 65 ksi	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

¾-in. Bolts

Bolt and Angle Available Strength, kips

5 Rows W30, 27, 24, 21, 18	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness ¼ ASD	¼ LRFD	5/16 ASD	5/16 LRFD	⅜ ASD	⅜ LRFD	½ ASD	½ LRFD
	A325/ F1852	N	—	83.3	125	104	156	106	159	106	159
		X	—	83.3	125	104	156	125	187	133	199
		SC Class A	STD	73.8	111	73.8	111	73.8	111	73.8	111
			OVS	53.3	80.0	53.3	80.0	53.3	80.0	53.3	80.0
			SSLT	62.8	94.1	62.8	94.1	62.8	94.1	62.8	94.1
		SC Class B	STD	83.3	125	104	156	105	158	105	158
			OVS	76.2	114	76.2	114	76.2	114	76.2	114
			SSLT	82.0	123	89.6	134	89.6	134	89.6	134
	A490	N	—	83.3	125	104	156	125	187	133	199
		X	—	83.3	125	104	156	125	187	166	249
		SC Class A	STD	83.3	125	92.3	138	92.3	138	92.3	138
			OVS	66.7	100	66.7	100	66.7	100	66.7	100
			SSLT	78.4	118	78.4	118	78.4	118	78.4	118
		SC Class B	STD	83.3	125	104	156	125	187	132	198
			OVS	82.4	124	95.2	143	95.2	143	95.2	143
			SSLT	82.0	123	102	154	112	168	112	168

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1½		1¾		1½		1¾		1½		1¾	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	208	312	216	324	195	293	203	305	205	307	213	320
	1⅜	210	316	219	328	197	296	206	308	207	311	216	323
	1½	213	319	221	332	200	300	208	312	210	315	218	327
	1⅝	215	323	223	335	202	303	210	316	212	318	220	331
	2	223	334	231	346	210	314	218	327	220	329	228	342
	3	242	363	250	375	229	344	237	356	239	359	247	371
Coped at Both Flanges	1¼	197	296	197	296	185	278	185	278	197	296	197	296
	1⅜	202	303	202	303	190	285	190	285	202	303	202	303
	1½	207	311	207	311	195	293	195	293	207	311	207	311
	1⅝	212	318	212	318	200	300	200	300	212	318	212	318
	2	223	334	227	340	210	314	215	322	220	329	227	340
	3	242	363	250	375	229	344	237	356	239	359	247	371
Uncoped		293	439	293	439	293	439	293	439	293	439	293	439

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	**585**	**878**

Notes:

STD = Standard holes

OVS = Oversized holes

SSLT = Short-slotted holes transverse to direction of load

N = Threads included

X = Threads excluded

SC = Slip critical

* Tabulated values include ¼-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

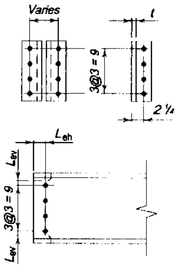
Beam F_y = 50 ksi, F_u = 65 ksi

Angle F_y = 36 ksi, F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows W24, 21, 18, 16	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	67.1	101	83.9	126	84.8	127	84.8	127
		X	—	67.1	101	83.9	126	101	151	106	159
		SC Class A	STD	59.1	88.6	59.1	88.6	59.1	88.6	59.1	88.6
			OVS	42.7	64.0	42.7	64.0	42.7	64.0	42.7	64.0
			SSLT	50.2	75.3	50.2	75.3	50.2	75.3	50.2	75.3
		SC Class B	STD	67.1	101	83.9	126	84.4	127	84.4	127
			OVS	61.0	91.4	61.0	91.4	61.0	91.4	61.0	91.4
			SSLT	65.8	98.7	71.7	108	71.7	108	71.7	108
	A490	N	—	67.1	101	83.9	126	101	151	106	159
		X	—	67.1	101	83.9	126	101	151	133	199
		SC Class A	STD	67.1	101	73.8	111	73.8	111	73.8	111
			OVS	53.3	80.0	53.3	80.0	53.3	80.0	53.3	80.0
			SSLT	62.8	94.1	62.8	94.1	62.8	94.1	62.8	94.1
		SC Class B	STD	67.1	101	83.9	126	101	151	105	158
			OVS	65.3	97.9	76.2	114	76.2	114	76.2	114
			SSLT	65.8	98.7	82.2	123	89.6	134	89.6	134

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
	L_{eh}*	1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
	L_{ev}, in.	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	167	250	175	262	156	234	164	246	164	245	172	257
	1 3/8	169	254	177	266	158	238	167	250	166	249	174	261
	1 1/2	171	257	180	269	161	241	169	254	168	253	177	265
	1 5/8	174	261	182	273	163	245	171	257	171	256	179	268
	2	181	272	189	284	171	256	179	268	178	267	186	279
	3	201	301	209	313	190	285	198	297	198	296	206	309
Coped at Both Flanges	1 1/4	156	234	156	234	146	219	146	219	156	234	156	234
	1 3/8	161	241	161	241	151	227	151	227	161	241	161	241
	1 1/2	166	249	166	249	156	234	156	234	166	249	166	249
	1 5/8	171	256	171	256	161	241	161	241	171	256	171	256
	2	181	272	185	278	171	256	176	263	178	267	185	278
	3	201	301	209	313	190	285	198	297	198	296	206	309
Uncoped		234	351	234	351	234	351	234	351	234	351	234	351

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	468	702

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

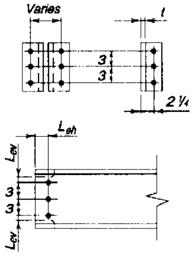
Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)

All-Bolted Double-Angle Connections

$3/4$-in. Bolts

Bolt and Angle Available Strength, kips

3 Rows W18, 16, 14, 12, 10⁺ ⁺Ltd. to W10x12, 15, 17, 19, 22, 26, 30	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness $1/4$ ASD	$1/4$ LRFD	$5/16$ ASD	$5/16$ LRFD	$3/8$ ASD	$3/8$ LRFD	$1/2$ ASD	$1/2$ LRFD
	A325/ F1852	N	—	50.9	76.4	63.6	95.4	63.6	95.4	63.6	95.4
		X	—	50.9	76.4	63.7	95.5	76.4	115	79.5	119
		SC Class A	STD	44.3	66.4	44.3	66.4	44.3	66.4	44.3	66.4
			OVS	32.0	48.0	32.0	48.0	32.0	48.0	32.0	48.0
			SSLT	37.7	56.5	37.7	56.5	37.7	56.5	37.7	56.5
		SC Class B	STD	50.9	76.4	63.3	94.9	63.3	94.9	63.3	94.9
			OVS	45.7	68.6	45.7	68.6	45.7	68.6	45.7	68.6
			SSLT	49.6	74.4	53.8	80.7	53.8	80.7	53.8	80.7
	A490	N	—	50.9	76.4	63.7	95.5	76.4	115	79.5	119
		X	—	50.9	76.4	63.7	95.5	76.4	115	99.4	149
		SC Class A	STD	50.9	76.4	55.4	83.1	55.4	83.1	55.4	83.1
			OVS	40.0	60.0	40.0	60.0	40.0	60.0	40.0	60.0
			SSLT	47.1	70.6	47.1	70.6	47.1	70.6	47.1	70.6
		SC Class B	STD	50.9	76.4	63.7	95.5	76.4	115	79.1	119
			OVS	47.9	71.8	57.1	85.7	57.1	85.7	57.1	85.7
			SSLT	49.6	74.4	62.0	92.9	67.2	101	67.2	101

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		$1\frac{1}{2}$		$1\frac{3}{4}$		$1\frac{1}{2}$		$1\frac{3}{4}$		$1\frac{1}{2}$		$1\frac{3}{4}$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	$1\frac{1}{4}$	125	188	133	200	117	176	125	188	122	183	130	195
	$1\frac{3}{8}$	128	191	136	204	119	179	128	191	125	187	133	199
	$1\frac{1}{2}$	130	195	138	207	122	183	130	195	127	190	135	203
	$1\frac{5}{8}$	132	199	141	211	124	186	132	199	129	194	138	206
	2	140	210	148	222	132	197	140	210	137	205	145	217
	3	159	239	167	251	151	227	159	239	156	234	164	246
Coped at Both Flanges	$1\frac{1}{4}$	115	172	115	172	107	161	107	161	115	172	115	172
	$1\frac{3}{8}$	119	179	119	179	112	168	112	168	119	179	119	179
	$1\frac{1}{2}$	124	186	124	186	117	176	117	176	124	186	124	186
	$1\frac{5}{8}$	129	194	129	194	122	183	122	183	129	194	129	194
	2	140	210	144	216	132	197	137	205	137	205	144	216
	3	159	239	167	251	151	227	159	239	156	234	164	246
Uncoped		175	263	175	263	175	263	175	263	175	263	175	263

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	351	526

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

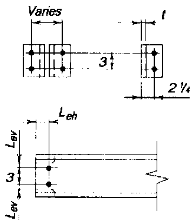
* Tabulated values include $1/4$-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

3/4-in. Bolts

Bolt and Angle Available Strength, kips

2 Rows	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
W12, 10, 8				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	32.6	48.9	40.8	61.2	42.4	63.6	42.4	63.6
		X	—	32.6	48.9	40.8	61.2	48.9	73.4	53.0	79.5
		SC Class A	STD	29.5	44.3	29.5	44.3	29.5	44.3	29.5	44.3
			OVS	21.3	32.0	21.3	32.0	21.3	32.0	21.3	32.0
			SSLT	25.1	37.7	25.1	37.7	25.1	37.7	25.1	37.7
		SC Class B	STD	32.6	48.9	40.8	61.2	42.2	63.3	42.2	63.3
			OVS	30.5	45.7	30.5	45.7	30.5	45.7	30.5	45.7
			SSLT	32.6	48.9	35.9	53.8	35.9	53.8	35.9	53.8
	A490	N	—	32.6	48.9	40.8	61.2	48.9	73.4	53.0	79.5
		X	—	32.6	48.9	40.8	61.2	48.9	73.4	65.3	97.9
		SC Class A	STD	32.6	48.9	36.9	55.4	36.9	55.4	36.9	55.4
			OVS	26.7	40.0	26.7	40.0	26.7	40.0	26.7	40.0
			SSLT	31.4	47.1	31.4	47.1	31.4	47.1	31.4	47.1
		SC Class B	STD	32.6	48.9	40.8	61.2	48.9	73.4	52.7	79.1
			OVS	30.5	45.7	38.1	57.1	38.1	57.1	38.1	57.1
			SSLT	32.6	48.9	40.8	61.2	44.8	67.2	44.8	67.2

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	83.7	126	91.4	137	78.0	117	86.1	129	80.6	121	88.8	133
	1 3/8	86.1	129	94.3	141	80.4	121	88.6	133	83.1	125	91.2	137
	1 1/2	88.6	133	96.7	145	82.9	124	91.0	137	85.5	128	93.6	140
	1 5/8	91.0	137	99.1	149	85.3	128	93.4	140	88.0	132	96.1	144
	2	98.3	147	106	160	92.6	139	101	151	95.3	143	103	155
	3	116	175	117	175	112	168	117	175	113	170	117	175
Coped at Both Flanges	1 1/4	73.1	110	73.1	110	68.3	102	68.3	102	73.1	110	73.1	110
	1 3/8	78.0	117	78.0	117	73.1	110	73.1	110	78.0	117	78.0	117
	1 1/2	82.9	124	82.9	124	78.0	117	78.0	117	82.9	124	82.9	124
	1 5/8	87.8	132	87.8	132	82.9	124	82.9	124	87.8	132	87.8	132
	2	98.3	147	102	154	92.6	139	97.5	146	95.3	143	102	154
	3	116	175	117	175	112	168	117	175	113	170	117	175
Uncoped		117	176	117	176	117	176	117	176	117	176	117	176

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	234	351

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi F_u = 65 ksi	F_y = 36 ksi F_u = 58 ksi

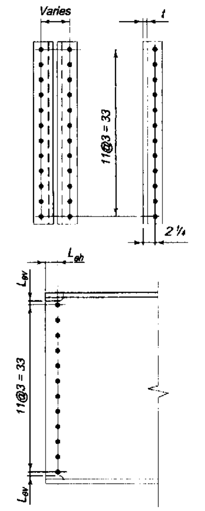
Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

12 Rows

W44



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	196	294	245	367	294	441	346	520
	X	—	196	294	245	367	294	441	392	587
	SC Class A	STD	196	294	245	367	247	370	247	370
		OVS	178	267	178	267	178	267	178	267
		SSLT	194	292	210	315	210	315	210	315
	SC Class B	STD	196	294	245	367	294	441	346	520
		OVS	191	287	239	359	255	382	255	382
		SSLT	194	292	243	365	292	438	300	450
A490	N	—	196	294	245	367	294	441	392	587
	X	—	196	294	245	367	294	441	392	587
	SC Class A	STD	196	294	245	367	294	441	310	465
		OVS	191	287	224	336	224	336	224	336
		SSLT	194	292	243	365	264	395	264	395
	SC Class B	STD	196	294	245	367	294	441	392	587
		OVS	191	287	239	359	287	431	320	480
		SSLT	194	292	243	365	292	438	377	565

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	468	702	476	714	438	657	446	669	465	697	473	710
	1 3/8	470	706	479	718	440	661	449	673	467	701	476	713
	1 1/2	473	709	481	722	443	664	451	676	470	705	478	717
	1 5/8	475	713	483	725	445	668	453	680	472	708	480	721
	2	483	724	491	736	453	679	461	691	480	719	488	732
	3	502	753	510	765	472	708	480	720	499	749	507	761
Coped at Both Flanges	1 1/4	458	687	458	687	429	644	429	644	458	687	458	687
	1 3/8	463	695	463	695	434	651	434	651	463	695	463	695
	1 1/2	468	702	468	702	439	658	439	658	468	702	468	702
	1 5/8	473	709	473	709	444	665	444	665	472	708	473	709
	2	483	724	488	731	453	679	458	687	480	719	488	731
	3	502	753	510	765	472	708	480	720	499	749	507	761
Uncoped		819	1230	819	1230	819	1230	819	1230	819	1230	819	1230

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1640	2460

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

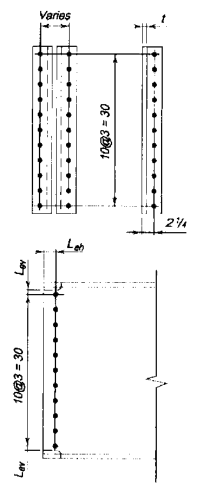
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

11 Rows	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
W44, 40				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	180	269	225	337	269	404	317	476
		X	—	180	269	225	337	269	404	359	539
		SC Class A	STD	180	269	225	337	226	339	226	339
			OVS	163	245	163	245	163	245	163	245
			SSLT	178	267	192	288	192	288	192	288
		SC Class B	STD	180	269	225	337	269	404	317	476
			OVS	175	263	219	328	233	350	233	350
			SSLT	178	267	223	334	267	401	275	412
	A490	N	—	180	269	225	337	269	404	359	539
		X	—	180	269	225	337	269	404	359	539
		SC Class A	STD	180	269	225	337	269	404	284	426
			OVS	175	263	205	308	205	308	205	308
			SSLT	178	267	223	334	242	362	242	362
		SC Class B	STD	180	269	225	337	269	404	359	539
			OVS	175	263	219	328	263	394	293	440
			SSLT	178	267	223	334	267	401	345	518

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	429	644	437	656	401	602	410	614	426	639	434	651
	1 3/8	431	647	440	659	404	606	412	618	428	643	437	655
	1 1/2	434	651	442	663	406	609	414	622	431	646	439	658
	1 5/8	436	654	444	667	409	613	417	625	433	650	441	662
	2	444	665	452	678	416	624	424	636	441	661	449	673
	3	463	695	471	707	436	653	444	665	460	690	468	702
Coped at Both Flanges	1 1/4	419	629	419	629	392	589	392	589	419	629	419	629
	1 3/8	424	636	424	636	397	596	397	596	424	636	424	636
	1 1/2	429	644	429	644	402	603	402	603	429	644	429	644
	1 5/8	434	651	434	651	407	611	407	611	433	650	434	651
	2	444	665	449	673	416	624	422	633	441	661	449	673
	3	463	695	471	707	436	653	444	665	460	690	468	702
Uncoped		751	1130	751	1130	751	1130	751	1130	751	1130	751	1130

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1500	2250

Notes:

STD = Standard holes

OVS = Oversized holes

SSLT = Short-slotted holes transverse to direction of load

N = Threads included

X = Threads excluded

SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

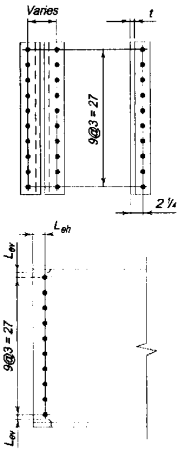
Beam F_y = 50 ksi
F_u = 65 ksi

Angle F_y = 36 ksi
F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

10 Rows W44, 40, 36	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	163	245	204	306	245	368	289	433
		X	—	163	245	204	306	245	368	327	490
		SC Class A	STD	163	245	204	306	206	308	206	308
			OVS	149	223	149	223	149	223	149	223
			SSLT	162	243	175	262	175	262	175	262
		SC Class B	STD	163	245	204	306	245	368	289	433
			OVS	159	238	198	298	212	318	212	318
			SSLT	162	243	203	304	243	365	250	375
	A490	N	—	163	245	204	306	245	368	327	490
		X	—	163	245	204	306	245	368	327	490
		SC Class A	STD	163	245	204	306	245	368	258	388
			OVS	159	238	187	280	187	280	187	280
			SSLT	162	243	203	304	220	329	220	329
		SC Class B	STD	163	245	204	306	245	368	327	490
			OVS	159	238	198	298	238	357	267	400
			SSLT	162	243	203	304	243	365	314	471

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	390	585	398	597	365	547	373	559	387	580	395	593
	1 3/8	392	589	401	601	367	551	375	563	389	584	398	596
	1 1/2	395	592	403	605	370	555	378	567	392	588	400	600
	1 5/8	397	596	405	608	372	558	380	570	394	591	402	604
	2	405	607	413	619	379	569	388	581	402	602	410	615
	3	424	636	432	648	399	598	407	611	421	632	429	644
Coped at Both Flanges	1 1/4	380	570	380	570	356	534	356	534	380	570	380	570
	1 3/8	385	578	385	578	361	541	361	541	385	578	385	578
	1 1/2	390	585	390	585	366	548	366	548	390	585	390	585
	1 5/8	395	592	395	592	371	556	371	556	394	591	395	592
	2	405	607	410	614	379	569	385	578	402	602	410	614
	3	424	636	432	648	399	598	407	611	421	632	429	644
Uncoped		683	1020	683	1020	683	1020	683	1020	683	1020	683	1020

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1370	2050

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam F_y = 50 ksi
F_u = 65 ksi

Angle F_y = 36 ksi
F_u = 58 ksi

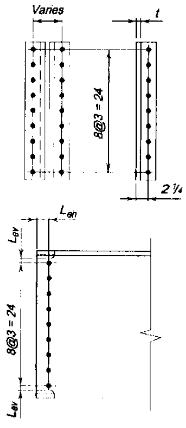
Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

9 Rows

W44, 40, 36, 33



ASTM Desig.	Thread Cond.	Hole Type	1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
A325/ F1852	N	—	147	221	184	276	221	331	260	390
	X	—	147	221	184	276	221	331	294	442
	SC Class A	STD	147	221	184	276	185	278	185	278
		OVS	134	201	134	201	134	201	134	201
		SSLT	146	219	157	236	157	236	157	236
	SC Class B	STD	147	221	184	276	221	331	260	390
		OVS	142	214	178	267	191	287	191	287
		SSLT	146	219	182	273	219	328	225	337
A490	N	—	147	221	184	276	221	331	294	442
	X	—	147	221	184	276	221	331	294	442
	SC Class A	STD	147	221	184	276	221	331	233	349
		OVS	142	214	168	252	168	252	168	252
		SSLT	146	219	182	273	198	297	198	297
	SC Class B	STD	147	221	184	276	221	331	294	442
		OVS	142	214	178	267	214	321	240	360
		SSLT	146	219	182	273	219	328	282	424

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	351	527	359	539	328	492	336	505	348	522	356	534
	1 3/8	353	530	362	542	331	496	339	508	350	526	359	538
	1 1/2	356	534	364	546	333	500	341	512	353	529	361	541
	1 5/8	358	537	366	550	336	503	344	516	355	533	363	545
	2	366	548	374	561	343	514	351	527	363	544	371	556
	3	385	578	393	590	362	544	371	556	382	573	390	585
Coped at Both Flanges	1 1/4	341	512	341	512	319	479	319	479	341	512	341	512
	1 3/8	346	519	346	519	324	486	324	486	346	519	346	519
	1 1/2	351	527	351	527	329	494	329	494	351	527	351	527
	1 5/8	356	534	356	534	334	501	334	501	355	533	356	534
	2	366	548	371	556	343	514	349	523	363	544	371	556
	3	385	578	393	590	362	544	371	556	382	573	390	585
Uncoped		614	921	614	921	614	921	614	921	614	921	614	921

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1230	1840

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

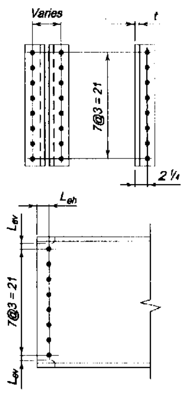
Beam F_y = 50 ksi
F_u = 65 ksi

Angle F_y = 36 ksi
F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

8 Rows W44, 40, 36, 33, 30	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	131	197	164	246	197	295	231	346
		X	—	131	197	164	246	197	295	262	393
		SC Class A	STD	131	197	164	246	165	247	165	247
			OVS	119	178	119	178	119	178	119	178
			SSLT	130	194	140	210	140	210	140	210
		SC Class B	STD	131	197	164	246	197	295	231	346
			OVS	126	189	158	237	170	255	170	255
			SSLT	130	194	162	243	194	292	200	300
	A490	N	—	131	197	164	246	197	295	262	393
		X	—	131	197	164	246	197	295	262	393
		SC Class A	STD	131	197	164	246	197	295	207	310
			OVS	126	189	149	224	149	224	149	224
			SSLT	130	194	162	243	176	264	176	264
		SC Class B	STD	131	197	164	246	197	295	262	393
			OVS	126	189	158	237	189	284	213	320
			SSLT	130	194	162	243	194	292	251	377

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1½		1¾		1½		1¾		1½		1¾	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	312	468	320	480	292	438	300	450	309	463	317	476
	1⅜	314	472	323	484	294	441	302	453	311	467	320	479
	1½	317	475	325	488	297	445	305	457	314	471	322	483
	1⅝	319	479	327	491	299	449	307	461	316	474	324	487
	2	327	490	335	502	306	459	314	472	324	485	332	498
	3	346	519	354	531	326	489	334	501	343	515	351	527
Coped at Both Flanges	1¼	302	453	302	453	283	424	283	424	302	453	302	453
	1⅜	307	461	307	461	288	431	288	431	307	461	307	461
	1½	312	468	312	468	293	439	293	439	312	468	312	468
	1⅝	317	475	317	475	297	446	297	446	316	474	317	475
	2	327	490	332	497	306	459	312	468	324	485	332	497
	3	346	519	354	531	326	489	334	501	343	515	351	527
Uncoped		546	819	546	819	546	819	546	819	546	819	546	819

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	1090	1640

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

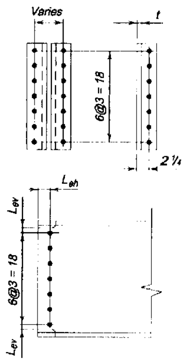
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi F_u = 65 ksi	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

$^7/_8$-in. Bolts

Bolt and Angle Available Strength, kips

7 Rows W44, 40, 36, 33, 30, 27, 24	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD	$^1/_2$ ASD	$^1/_2$ LRFD
	A325/ F1852	N	—	115	172	144	215	172	258	202	303
		X	—	115	172	144	215	172	258	230	344
		SC Class A	STD	115	172	144	215	144	216	144	216
			OVS	104	156	104	156	104	156	104	156
			SSLT	113	170	122	184	122	184	122	184
		SC Class B	STD	115	172	144	215	172	258	202	303
			OVS	110	165	137	206	149	223	149	223
			SSLT	113	170	142	213	170	255	175	262
	A490	N	—	115	172	144	215	172	258	230	344
		X	—	115	172	144	215	172	258	230	344
		SC Class A	STD	115	172	144	215	172	258	181	271
			OVS	110	165	131	196	131	196	131	196
			SSLT	113	170	142	213	154	231	154	231
		SC Class B	STD	115	172	144	215	172	258	230	344
			OVS	110	165	137	206	165	247	187	280
			SSLT	113	170	142	213	170	255	220	329

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		$1^1/_2$		$1^3/_4$		$1^1/_2$		$1^3/_4$		$1^1/_2$		$1^3/_4$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	$1^1/_4$	273	410	281	422	255	383	263	395	270	405	278	417
	$1^3/_8$	275	413	284	425	258	386	266	399	272	409	281	421
	$1^1/_2$	278	417	286	429	260	390	268	402	275	412	283	424
	$1^5/_8$	280	420	288	433	262	394	271	406	277	416	285	428
	2	288	431	296	444	270	405	278	417	285	427	293	439
	3	307	461	315	473	289	434	297	446	304	456	312	468
Coped at Both Flanges	$1^1/_4$	263	395	263	395	246	369	246	369	263	395	263	395
	$1^3/_8$	268	402	268	402	251	377	251	377	268	402	268	402
	$1^1/_2$	273	410	273	410	256	384	256	384	273	410	273	410
	$1^5/_8$	278	417	278	417	261	391	261	391	277	416	278	417
	2	288	431	293	439	270	405	275	413	285	427	293	439
	3	307	461	315	473	289	434	297	446	304	456	312	468
Uncoped		478	717	478	717	478	717	478	717	478	717	478	717

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	956	1430

Notes:

STD = Standard holes

OVS = Oversized holes

SSLT = Short-slotted holes transverse to direction of load

N = Threads included

X = Threads excluded

SC = Slip critical

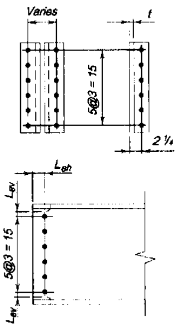
* Tabulated values include $^1/_4$-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

6 Rows W40, 36, 33, 30, 27, 24, 21	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	**98.6**	148	**123**	185	**148**	222	**173**	260
		X	—	**98.6**	148	**123**	185	**148**	222	**197**	296
		SC Class A	STD	**98.6**	148	**123**	185	**123**	185	**123**	185
			OVS	**89.2**	134	**89.2**	134	**89.2**	134	**89.2**	134
			SSLT	**97.3**	146	**105**	157	**105**	157	**105**	157
		SC Class B	STD	**98.6**	148	**123**	185	**148**	222	**173**	260
			OVS	**93.5**	140	**117**	175	**127**	191	**127**	191
			SSLT	**97.3**	146	**122**	182	**146**	219	**150**	225
	A490	N	—	**98.6**	148	**123**	185	**148**	222	**197**	296
		X	—	**98.6**	148	**123**	185	**148**	222	**197**	296
		SC Class A	STD	**98.6**	148	**123**	185	**148**	222	**155**	233
			OVS	**93.5**	140	**112**	168	**112**	168	**112**	168
			SSLT	**97.3**	146	**122**	182	**132**	198	**132**	198
		SC Class B	STD	**98.6**	148	**123**	185	**148**	222	**197**	296
			OVS	**93.5**	140	**117**	175	**140**	210	**160**	240
			SSLT	**97.3**	146	**122**	182	**146**	219	**188**	282

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1½		1¾		1½		1¾		1½		1¾	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	**234**	351	**242**	363	**219**	328	**227**	340	**231**	346	**239**	359
	1⅜	**236**	355	**245**	367	**221**	332	**229**	344	**233**	350	**242**	362
	1½	**239**	358	**247**	371	**223**	335	**232**	347	**236**	354	**244**	366
	1⅝	**241**	362	**249**	374	**226**	339	**234**	351	**238**	357	**246**	370
	2	**249**	373	**257**	385	**233**	350	**241**	362	**246**	368	**254**	381
	3	**268**	402	**276**	414	**253**	379	**261**	391	**265**	398	**273**	410
Coped at Both Flanges	1¼	**224**	336	**224**	336	**210**	314	**210**	314	**224**	336	**224**	336
	1⅜	**229**	344	**229**	344	**215**	322	**215**	322	**229**	344	**229**	344
	1½	**234**	351	**234**	351	**219**	329	**219**	329	**234**	351	**234**	351
	1⅝	**239**	358	**239**	358	**224**	336	**224**	336	**238**	357	**239**	358
	2	**249**	373	**254**	380	**233**	350	**239**	358	**246**	368	**254**	380
	3	**268**	402	**276**	414	**253**	379	**261**	391	**265**	398	**273**	410
Uncoped		**409**	614	**409**	614	**409**	614	**409**	614	**409**	614	**409**	614

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	**819**	1230

Notes:

STD = Standard holes

OVS = Oversized holes

SSLT = Short-slotted holes transverse to direction of load

N = Threads included

X = Threads excluded

SC = Slip critical

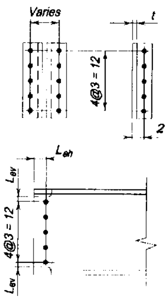
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

5 Rows W30, 27, 24, 21, 18	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	82.4	124	103	155	124	185	144	216
		X	—	82.4	124	103	155	124	185	165	247
		SC Class A	STD	82.4	124	103	154	103	154	103	154
			OVS	74.3	111	74.3	111	74.3	111	74.3	111
			SSLT	81.1	122	87.4	131	87.4	131	87.4	131
		SC Class B	STD	82.4	124	103	155	124	185	144	216
			OVS	77.2	116	96.5	145	106	159	106	159
			SSLT	81.1	122	101	152	122	182	125	187
	A490	N	—	82.4	124	103	155	124	185	165	247
		X	—	82.4	124	103	155	124	185	165	247
		SC Class A	STD	82.4	124	103	155	124	185	129	194
			OVS	77.2	116	93.3	140	93.3	140	93.3	140
			SSLT	81.1	122	101	152	110	165	110	165
		SC Class B	STD	82.4	124	103	155	124	185	165	247
			OVS	77.2	116	96.5	145	116	174	133	200
			SSLT	81.1	122	101	152	122	182	157	235

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	195	293	203	305	182	273	190	285	192	288	200	300
	1 3/8	197	296	206	308	184	277	193	289	194	292	203	304
	1 1/2	200	300	208	312	187	280	195	293	197	295	205	307
	1 5/8	202	303	210	316	189	284	197	296	199	299	207	311
	2	210	314	218	327	197	295	205	307	207	310	215	322
	3	229	344	237	356	216	324	224	336	226	339	234	351
Coped at Both Flanges	1 1/4	185	278	185	278	173	260	173	260	185	278	185	278
	1 3/8	190	285	190	285	178	267	178	267	190	285	190	285
	1 1/2	195	293	195	293	183	274	183	274	195	293	195	293
	1 5/8	200	300	200	300	188	282	188	282	199	299	200	300
	2	210	314	215	322	197	295	202	303	207	310	215	322
	3	229	344	237	356	216	324	224	336	226	339	234	351
Uncoped		341	512	341	512	341	512	341	512	341	512	341	512

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	683	1020

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

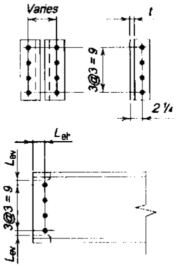
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows W24, 21, 18, 16	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	65.3	97.9	81.6	122	97.9	147	115	173
		X	—	65.3	97.9	81.6	122	97.9	147	131	196
		SC Class A	STD	65.3	97.9	81.6	122	82.3	123	82.3	123
			OVS	59.4	89.2	59.4	89.2	59.4	89.2	59.4	89.2
			SSLT	64.9	97.3	69.9	105	69.9	105	69.9	105
		SC Class B	STD	65.3	97.9	81.6	122	97.9	147	115	173
			OVS	60.9	91.4	76.1	114	84.9	127	84.9	127
			SSLT	64.9	97.3	81.1	122	97.3	146	99.9	150
	A490	N	—	65.3	97.9	81.6	122	97.9	147	131	196
		X	—	65.3	97.9	81.6	122	97.9	147	131	196
		SC Class A	STD	65.3	97.9	81.6	122	97.9	147	103	155
			OVS	60.9	91.4	74.7	112	74.7	112	74.7	112
			SSLT	64.9	97.3	81.1	122	87.9	132	87.9	132
		SC Class B	STD	65.3	97.9	81.6	122	97.9	147	131	196
			OVS	60.9	91.4	76.1	114	91.4	137	107	160
			SSLT	64.9	97.3	81.1	122	97.3	146	126	188

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	156	234	164	246	145	218	154	230	153	229	161	242
	1 3/8	158	238	167	250	148	222	156	234	155	233	164	245
	1 1/2	161	241	169	254	150	225	158	238	158	237	166	249
	1 5/8	163	245	171	257	153	229	161	241	160	240	168	253
	2	171	256	179	268	160	240	168	252	168	251	176	264
	3	190	285	198	297	180	269	188	282	187	281	195	293
Coped at Both Flanges	1 1/4	146	219	146	219	137	205	137	205	146	219	146	219
	1 3/8	151	227	151	227	141	212	141	212	151	227	151	227
	1 1/2	156	234	156	234	146	219	146	219	156	234	156	234
	1 5/8	161	241	161	241	151	227	151	227	160	240	161	241
	2	171	256	176	263	160	240	166	249	168	251	176	263
	3	190	285	198	297	180	269	188	282	187	281	195	293
Uncoped		273	410	273	410	273	410	273	410	273	410	273	410

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	546	819

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

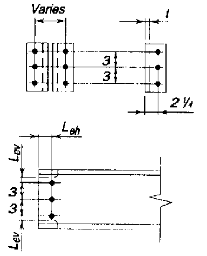
Beam F_y = 50 ksi, F_u = 65 ksi

Angle F_y = 36 ksi, F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

7/8-in. Bolts

Bolt and Angle Available Strength, kips

3 Rows W18, 16, 14, 12, 10⁺ ⁺Ltd. to W10x12, 15, 17, 19, 22, 26, 30	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	47.9	71.8	59.8	89.7	71.8	108	86.6	130
		X	—	47.9	71.8	59.8	89.7	71.8	108	95.7	144
		SC Class A	STD	47.9	71.8	59.8	89.7	61.7	92.5	61.7	92.5
			OVS	44.6	66.9	44.6	66.9	44.6	66.9	44.6	66.9
			SSLT	47.9	71.8	52.4	78.7	52.4	78.7	52.4	78.7
		SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108	86.6	130
			OVS	44.6	66.9	55.7	83.6	63.7	95.5	63.7	95.5
			SSLT	47.9	71.8	59.8	89.7	71.8	108	74.9	112
	A490	N	—	47.9	71.8	59.8	89.7	71.8	108	95.7	144
		X	—	47.9	71.8	59.8	89.7	71.8	108	95.7	144
		SC Class A	STD	47.9	71.8	59.8	89.7	71.8	108	77.5	116
			OVS	44.6	66.9	55.7	83.6	56	84	56	84
			SSLT	47.9	71.8	59.8	89.7	65.9	98.8	65.9	98.8
		SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108	95.7	144
			OVS	44.6	66.9	55.7	83.6	66.9	100	80	120
			SSLT	47.9	71.8	59.8	89.7	71.8	108	94.1	141

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	117	176	125	188	109	163	117	176	114	171	122	183
	1 3/8	119	179	128	191	111	167	119	179	116	175	125	187
	1 1/2	122	183	130	195	114	171	122	183	119	178	127	190
	1 5/8	124	186	132	199	116	174	124	186	121	182	129	194
	2	132	197	140	210	124	185	132	197	129	193	137	205
	3	151	227	159	239	143	215	151	227	148	222	156	234
Coped at Both Flanges	1 1/4	107	161	107	161	99.9	150	99.9	150	107	161	107	161
	1 3/8	112	168	112	168	105	157	105	157	112	168	112	168
	1 1/2	117	176	117	176	110	165	110	165	117	176	117	176
	1 5/8	122	183	122	183	115	172	115	172	121	182	122	183
	2	132	197	137	205	124	185	129	194	129	193	137	205
	3	151	227	159	239	143	215	151	227	148	222	156	234
Uncoped		205	307	205	307	205	307	205	307	205	307	205	307

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	409	614

Notes:

STD = Standard holes	N = Threads included
OVS = Oversized holes	X = Threads excluded
SSLT = Short-slotted holes transverse to direction of load	SC = Slip critical

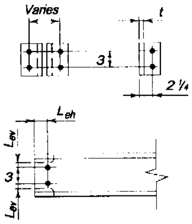
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	
Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

$^7/_8$-in. Bolts

Bolt and Angle Available Strength, kips

2 Rows W12, 10, 8	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD	$^1/_2$ ASD	$^1/_2$ LRFD
	A325/ F1852	N	—	30.5	45.7	38.1	57.1	45.7	68.5	57.7	86.6
		X	—	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
		SC Class A	STD	30.5	45.7	38.1	57.1	41.1	61.7	41.1	61.7
			OVS	28.3	42.4	29.7	44.6	29.7	44.6	29.7	44.6
			SSLT	30.5	45.7	35.0	52.4	35.0	52.4	35.0	52.4
		SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5	57.7	86.6
			OVS	28.3	42.4	35.3	53.0	42.4	63.6	42.5	63.7
			SSLT	30.5	45.7	38.1	57.1	45.7	68.5	49.9	74.9
	A490	N	—	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
		X	—	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
		SC Class A	STD	30.5	45.7	38.1	57.1	45.7	68.5	51.7	77.5
			OVS	28.3	42.4	35.3	53.0	37.3	56.0	37.3	56.0
			SSLT	30.5	45.7	38.1	57.1	43.9	65.9	43.9	65.9
		SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4
			OVS	28.3	42.4	35.3	53.0	42.4	63.6	53.3	80.0
			SSLT	30.5	45.7	38.1	57.1	45.7	68.5	60.9	91.4

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		$1^1/_2$		$1^3/_4$		$1^1/_2$		$1^3/_4$		$1^1/_2$		$1^3/_4$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	$1^1/_4$	78.0	117	86.1	129	72.3	108	80.4	121	75.0	112	83.1	125
	$1^3/_8$	80.4	121	88.6	133	74.8	112	82.9	124	77.4	116	85.5	128
	$1^1/_2$	82.9	124	91.0	137	77.2	116	85.3	128	79.8	120	88.0	132
	$1^5/_8$	85.3	128	93.4	140	79.6	119	87.8	132	82.3	123	90.4	136
	2	92.6	139	101	151	86.9	130	95.1	143	89.6	134	97.7	147
	3	112	168	120	180	106	160	115	172	109	164	117	176
Coped at Both Flanges	$1^1/_4$	68.3	102	68.3	102	63.4	95.1	63.4	95.1	68.3	102	68.3	102
	$1^3/_8$	73.1	110	73.1	110	68.3	102	68.3	102	73.1	110	73.1	110
	$1^1/_2$	78.0	117	78.0	117	73.1	110	73.1	110	78.0	117	78.0	117
	$1^5/_8$	82.9	124	82.9	124	78.0	117	78.0	117	82.3	123	82.9	124
	2	92.6	139	97.5	146	86.9	130	92.6	139	89.6	134	97.5	146
	3	112	168	120	180	106	160	115	172	109	164	117	176
Uncoped		137	205	137	205	137	205	137	205	137	205	137	205

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ OVS/ SSLT	273	410

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

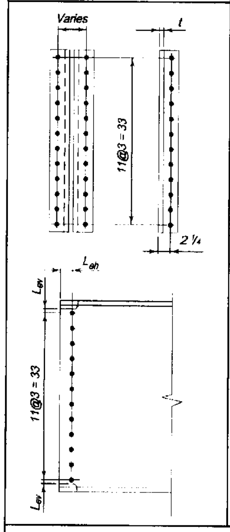
* Tabulated values include $^1/_4$-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi F_u = 65 ksi	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips



12 Rows W44	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	191	287	239	359	287	431	383	574
		X	—	191	287	239	359	287	431	383	574
		SC Class A	STD	191	287	239	359	287	431	323	484
			OVS	172	258	215	322	233	350	233	350
			SSLT	191	287	239	359	274	411	274	411
		SC Class B	STD	191	287	239	359	287	431	383	574
			OVS	172	258	215	322	258	387	333	500
			SSLT	191	287	239	359	287	431	383	574
	A490	N	—	191	287	239	359	287	431	383	574
		X	—	191	287	239	359	287	431	383	574
		SC Class A	STD	191	287	239	359	287	431	383	574
			OVS	172	258	215	322	258	387	293	439
			SSLT	191	287	239	359	287	431	344	516
		SC Class B	STD	191	287	239	359	287	431	383	574
			OVS	172	258	215	322	258	387	344	515
			SSLT	191	287	239	359	287	431	383	574

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}* 1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
L_{ev}, in.		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	438	657	446	669	393	589	401	601	434	651	442	663
	1 3/8	440	661	449	673	395	593	403	605	436	654	444	667
	1 1/2	443	664	451	676	398	597	406	609	439	658	447	670
	1 5/8	445	668	453	680	400	600	408	612	441	662	449	674
	2	453	679	461	691	407	611	416	623	449	673	457	685
	3	472	708	480	720	427	640	435	653	468	702	476	714
Coped at Both Flanges	1 1/4	429	644	429	644	385	578	385	578	429	644	429	644
	1 3/8	434	651	434	651	390	585	390	585	434	651	434	651
	1 1/2	439	658	439	658	395	592	395	592	439	658	439	658
	1 5/8	444	665	444	665	400	600	400	600	441	662	444	665
	2	453	679	458	687	407	611	414	622	449	673	457	685
	3	472	708	480	720	427	640	435	653	468	702	476	714
Uncoped		909	1360	909	1360	829	1240	829	1240	909	1360	909	1360

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1820	2730
OVS	1660	2490

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

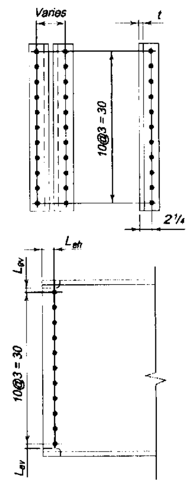
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

11 Rows W44, 40	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	175	263	219	328	263	394	350	525
		X	—	175	263	219	328	263	394	350	525
		SC Class A	STD	175	263	219	328	263	394	296	444
			OVS	157	236	196	295	214	321	214	321
			SSLT	175	263	219	328	251	377	251	377
		SC Class B	STD	175	263	219	328	263	394	350	525
			OVS	157	236	196	295	236	354	305	458
			SSLT	175	263	219	328	263	394	350	525
	A490	N	—	175	263	219	328	263	394	350	525
		X	—	175	263	219	328	263	394	350	525
		SC Class A	STD	175	263	219	328	263	394	350	525
			OVS	157	236	196	295	236	354	268	402
			SSLT	175	263	219	328	263	394	316	473
		SC Class B	STD	175	263	219	328	263	394	350	525
			OVS	157	236	196	295	236	354	314	471
			SSLT	175	263	219	328	263	394	350	525

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1½		1¾		1½		1¾		1½		1¾	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1¼	401	602	410	614	360	540	368	552	397	596	405	608
	1⅜	404	606	412	618	362	544	371	556	400	600	408	612
	1½	406	609	414	622	365	547	373	559	402	603	410	615
	1⅝	409	613	417	625	367	551	375	563	405	607	413	619
	2	416	624	424	636	375	562	383	574	412	618	420	630
	3	436	653	444	665	394	591	402	603	431	647	440	659
Coped at Both Flanges	1¼	392	589	392	589	352	528	352	528	392	589	392	589
	1⅜	397	596	397	596	357	536	357	536	397	596	397	596
	1½	402	603	402	603	362	543	362	543	402	603	402	603
	1⅝	407	611	407	611	367	550	367	550	405	607	407	611
	2	416	624	422	633	375	562	381	572	412	618	420	630
	3	436	653	444	665	394	591	402	603	431	647	440	659
Uncoped		834	1250	834	1250	761	1140	761	1140	834	1250	834	1250

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1670	2500
OVS	1520	2280

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

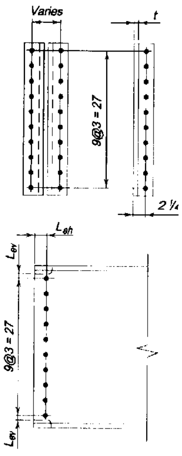
* Tabulated values include ¼-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

10 Rows W44, 40, 36	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	159	238	198	298	238	357	318	476
		X	—	159	238	198	298	238	357	318	476
		SC Class A	STD	159	238	198	298	238	357	269	403
			OVS	142	214	178	267	194	291	194	291
			SSLT	159	238	198	298	229	343	229	343
		SC Class B	STD	159	238	198	298	238	357	318	476
			OVS	142	214	178	267	214	321	278	416
			SSLT	159	238	198	298	238	357	318	476
	A490	N	—	159	238	198	298	238	357	318	476
		X	—	159	238	198	298	238	357	318	476
		SC Class A	STD	159	238	198	298	238	357	318	476
			OVS	142	214	178	267	214	321	244	366
			SSLT	159	238	198	298	238	357	287	430
		SC Class B	STD	159	238	198	298	238	357	318	476
			OVS	142	214	178	267	214	321	285	427
			SSLT	159	238	198	298	238	357	318	476

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	365	547	373	559	327	491	335	503	361	541	369	553
	1 3/8	367	551	375	563	329	494	338	506	363	545	371	557
	1 1/2	370	555	378	567	332	498	340	510	366	548	374	561
	1 5/8	372	558	380	570	334	502	342	514	368	552	376	564
	2	379	569	388	581	342	512	350	525	375	563	384	575
	3	399	598	407	611	361	542	369	554	395	592	403	605
Coped at Both Flanges	1 1/4	356	534	356	534	319	479	319	479	356	534	356	534
	1 3/8	361	541	361	541	324	486	324	486	361	541	361	541
	1 1/2	366	548	366	548	329	494	329	494	366	548	366	548
	1 5/8	371	556	371	556	334	501	334	501	368	552	371	556
	2	379	569	385	578	342	512	349	523	375	563	384	575
	3	399	598	407	611	361	542	369	554	395	592	403	605
Uncoped		758	1140	758	1140	692	1040	692	1040	758	1140	758	1140

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1520	2270
OVS	1380	2080

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

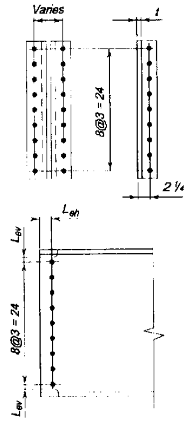
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	
Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

9 Rows	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
W44, 40, 36, 33				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	142	214	178	267	214	321	285	427
		X	—	142	214	178	267	214	321	285	427
		SC Class A	STD	142	214	178	267	214	321	242	363
			OVS	128	192	160	240	175	262	175	262
			SSLT	142	214	178	267	206	309	206	309
		SC Class B	STD	142	214	178	267	214	321	285	427
			OVS	128	192	160	240	192	288	250	375
			SSLT	142	214	178	267	214	321	285	427
	A490	N	—	142	214	178	267	214	321	285	427
		X	—	142	214	178	267	214	321	285	427
		SC Class A	STD	142	214	178	267	214	321	285	427
			OVS	128	192	160	240	192	288	219	329
			SSLT	142	214	178	267	214	321	258	387
		SC Class B	STD	142	214	178	267	214	321	285	427
			OVS	128	192	160	240	192	288	256	383
			SSLT	142	214	178	267	214	321	285	427

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	328	492	336	505	294	441	302	453	324	486	332	498
	1 3/8	331	496	339	508	297	445	305	457	327	490	335	502
	1 1/2	333	500	341	512	299	449	307	461	329	494	337	506
	1 5/8	336	503	344	516	301	452	310	464	332	497	340	509
	2	343	514	351	527	309	463	317	475	339	508	347	520
	3	362	544	371	556	328	492	336	505	358	537	366	550
Coped at Both Flanges	1 1/4	319	479	319	479	286	430	286	430	319	479	319	479
	1 3/8	324	486	324	486	291	437	291	437	324	486	324	486
	1 1/2	329	494	329	494	296	444	296	444	329	494	329	494
	1 5/8	334	501	334	501	301	452	301	452	332	497	334	501
	2	343	514	349	523	309	463	316	473	339	508	347	520
	3	362	544	371	556	328	492	336	505	358	537	366	550
Uncoped		683	1020	683	1020	624	936	624	936	683	1020	683	1020

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1370	2050
OVS	1250	1870

Notes:

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

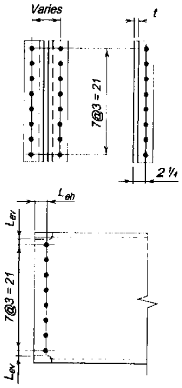
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

8 Rows W44, 40, 36, 33, 30	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	126	189	158	237	189	284	252	378
		X	—	126	189	158	237	189	284	252	378
		SC Class A	STD	126	189	158	237	189	284	215	323
			OVS	113	170	141	212	155	233	155	233
			SSLT	126	189	158	237	183	274	183	274
		SC Class B	STD	126	189	158	237	189	284	252	378
			OVS	113	170	141	212	170	254	222	333
			SSLT	126	189	158	237	189	284	252	378
	A490	N	—	126	189	158	237	189	284	252	378
		X	—	126	189	158	237	189	284	252	378
		SC Class A	STD	126	189	158	237	189	284	252	378
			OVS	113	170	141	212	170	254	195	293
			SSLT	126	189	158	237	189	284	229	344
		SC Class B	STD	126	189	158	237	189	284	252	378
			OVS	113	170	141	212	170	254	226	339
			SSLT	126	189	158	237	189	284	252	378

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	292	438	300	450	261	392	269	404	288	431	296	444
	1 3/8	294	441	302	453	264	395	272	408	290	435	298	447
	1 1/2	297	445	305	457	266	399	274	411	293	439	301	451
	1 5/8	299	449	307	461	269	403	277	415	295	442	303	455
	2	306	459	314	472	276	414	284	426	302	453	310	466
	3	326	489	334	501	295	443	303	455	322	483	330	495
Coped at Both Flanges	1 1/4	283	424	283	424	254	380	254	380	283	424	283	424
	1 3/8	288	431	288	431	258	388	258	388	288	431	288	431
	1 1/2	293	439	293	439	263	395	263	395	293	439	293	439
	1 5/8	297	446	297	446	268	402	268	402	295	442	297	446
	2	306	459	312	468	276	414	283	424	302	453	310	466
	3	326	489	334	501	295	443	303	455	322	483	330	495
Uncoped		607	910	607	910	556	834	556	834	607	910	607	910

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1210	1820
OVS	1110	1670

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

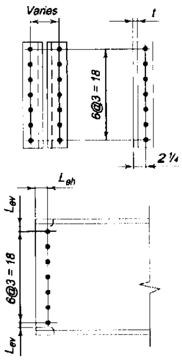
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi F_u = 65 ksi	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

7 Rows W44, 40, 36, 33, 30, 27, 24	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	110	165	137	206	165	247	220	330
		X	—	110	165	137	206	165	247	220	330
		SC Class A	STD	110	165	137	206	165	247	188	282
			OVS	98.4	148	123	185	136	204	136	204
			SSLT	110	165	137	206	160	240	160	240
		SC Class B	STD	110	165	137	206	165	247	220	330
			OVS	98.4	148	123	185	148	221	194	291
			SSLT	110	165	137	206	165	247	220	330
	A490	N	—	110	165	137	206	165	247	220	330
		X	—	110	165	137	206	165	247	220	330
		SC Class A	STD	110	165	137	206	165	247	220	330
			OVS	98.4	148	123	185	148	221	171	256
			SSLT	110	165	137	206	165	247	201	301
		SC Class B	STD	110	165	137	206	165	247	220	330
			OVS	98.4	148	123	185	148	221	197	295
			SSLT	110	165	137	206	165	247	220	330

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	255	383	263	395	228	342	236	355	251	377	259	389
	1 3/8	258	386	266	399	231	346	239	358	254	380	262	392
	1 1/2	260	390	268	402	233	350	241	362	256	384	264	396
	1 5/8	262	394	271	406	236	353	244	366	258	388	267	400
	2	270	405	278	417	243	364	251	377	266	399	274	411
	3	289	434	297	446	262	394	271	406	285	428	293	440
Coped at Both Flanges	1 1/4	246	369	246	369	221	331	221	331	246	369	246	369
	1 3/8	251	377	251	377	225	338	225	338	251	377	251	377
	1 1/2	256	384	256	384	230	346	230	346	256	384	256	384
	1 5/8	261	391	261	391	235	353	235	353	258	388	261	391
	2	270	405	275	413	243	364	250	375	266	399	274	411
	3	289	434	297	446	262	394	271	406	285	428	293	440
Uncoped		531	797	531	797	487	731	487	731	531	797	531	797

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	1060	1590
OVS	975	1460

Notes:

STD = Standard holes

OVS = Oversized holes

SSLT = Short-slotted holes transverse to direction of load

N = Threads included

X = Threads excluded

SC = Slip critical

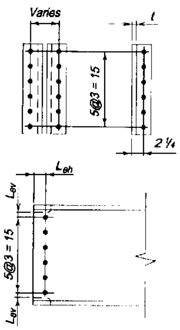
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	Angle
F_y = 50 ksi F_u = 65 ksi	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

6 Rows W40, 36, 33, 30, 27, 24, 21	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	93.5	140	117	175	140	210	187	281
		X	—	93.5	140	117	175	140	210	187	281
		SC Class A	STD	93.5	140	117	175	140	210	161	242
			OVS	83.7	126	105	157	117	175	117	175
			SSLT	93.5	140	117	175	137	206	137	206
		SC Class B	STD	93.5	140	117	175	140	210	187	281
			OVS	83.7	126	105	157	126	188	167	250
			SSLT	93.5	140	117	175	140	210	187	281
	A490	N	—	93.5	140	117	175	140	210	187	281
		X	—	93.5	140	117	175	140	210	187	281
		SC Class A	STD	93.5	140	117	175	140	210	187	281
			OVS	83.7	126	105	157	126	188	146	219
			SSLT	93.5	140	117	175	140	210	172	258
		SC Class B	STD	93.5	140	117	175	140	210	187	281
			OVS	83.7	126	105	157	126	188	167	251
			SSLT	93.5	140	117	175	140	210	187	281

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	219	328	227	340	195	293	204	305	215	322	223	334
	1 3/8	221	332	229	344	198	297	206	309	217	325	225	338
	1 1/2	223	335	232	347	200	300	208	313	219	329	228	341
	1 5/8	226	339	234	351	203	304	211	316	222	333	230	345
	2	233	350	241	362	210	315	218	327	229	344	237	356
	3	253	379	261	391	230	344	238	356	249	373	257	385
Coped at Both Flanges	1 1/4	210	314	210	314	188	282	188	282	210	314	210	314
	1 3/8	215	322	215	322	193	289	193	289	215	322	215	322
	1 1/2	219	329	219	329	197	296	197	296	219	329	219	329
	1 5/8	224	336	224	336	202	303	202	303	222	333	224	336
	2	233	350	239	358	210	315	217	325	229	344	237	356
	3	253	379	261	391	230	344	238	356	249	373	257	385
Uncoped		456	684	456	684	419	629	419	629	456	684	456	684

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	912	1370
OVS	839	1260

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

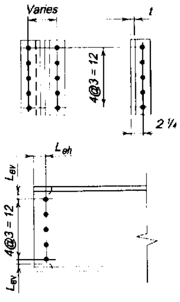
Beam F_y = 50 ksi, F_u = 65 ksi

Angle F_y = 36 ksi, F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

5 Rows W30, 27, 24, 21, 18	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	77.2	116	96.5	145	116	174	154	232
		X	—	77.2	116	96.5	145	116	174	154	232
		SC Class A	STD	77.2	116	96.5	145	116	174	134	202
			OVS	69.1	104	86.3	129	97.2	146	97.2	146
			SSLT	77.2	116	96.5	145	114	171	114	171
		SC Class B	STD	77.2	116	96.5	145	116	174	154	232
			OVS	69.1	104	86.3	129	104	155	138	207
			SSLT	77.2	116	96.5	145	116	174	154	232
	A490	N	—	77.2	116	96.5	145	116	174	154	232
		X	—	77.2	116	96.5	145	116	174	154	232
		SC Class A	STD	77.2	116	96.5	145	116	174	154	232
			OVS	69.1	104	86.3	129	104	155	122	183
			SSLT	77.2	116	96.5	145	116	174	143	215
		SC Class B	STD	77.2	116	96.5	145	116	174	154	232
			OVS	69.1	104	86.3	129	104	155	138	207
			SSLT	77.2	116	96.5	145	116	174	154	232

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	182	273	190	285	163	244	171	256	178	267	186	279
	1 3/8	184	277	193	289	165	247	173	260	180	271	189	283
	1 1/2	187	280	195	293	167	251	176	263	183	274	191	286
	1 5/8	189	284	197	296	170	255	178	267	185	278	193	290
	2	197	295	205	307	177	266	185	278	193	289	201	301
	3	216	324	224	336	197	295	205	307	212	318	220	330
Coped at Both Flanges	1 1/4	173	260	173	260	155	232	155	232	173	260	173	260
	1 3/8	178	267	178	267	160	239	160	239	178	267	178	267
	1 1/2	183	274	183	274	165	247	165	247	183	274	183	274
	1 5/8	188	282	188	282	169	254	169	254	185	278	188	282
	2	197	295	202	303	177	266	184	276	193	289	201	301
	3	216	324	224	336	197	295	205	307	212	318	220	330
Uncoped		380	570	380	570	351	527	351	527	380	570	380	570

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	761	1140
OVS	702	1050

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

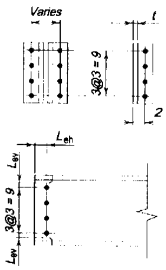
Beam F_y = 50 ksi
F_u = 65 ksi

Angle F_y = 36 ksi
F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

4 Rows	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
W24, 21, 18, 16				$^1/_4$		$^5/_{16}$		$^3/_8$		$^1/_2$	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	60.9	91.4	76.1	114	91.4	137	122	183
		X	—	60.9	91.4	76.1	114	91.4	137	122	183
		SC Class A	STD	60.9	91.4	76.1	114	91.4	137	108	161
			OVS	54.4	81.6	68.0	102	77.7	117	77.7	117
			SSLT	60.9	91.4	76.1	114	91.4	137	91.4	137
		SC Class B	STD	60.9	91.4	76.1	114	91.4	137	122	183
			OVS	54.4	81.6	68.0	102	81.6	122	109	163
			SSLT	60.9	91.4	76.1	114	91.4	137	122	183
	A490	N	—	60.9	91.4	76.1	114	91.4	137	122	183
		X	—	60.9	91.4	76.1	114	91.4	137	122	183
		SC Class A	STD	60.9	91.4	76.1	114	91.4	137	122	183
			OVS	54.4	81.6	68.0	102	81.6	122	97.5	146
			SSLT	60.9	91.4	76.1	114	91.4	137	115	172
		SC Class B	STD	60.9	91.4	76.1	114	91.4	137	122	183
			OVS	54.4	81.6	68.0	102	81.6	122	109	163
			SSLT	60.9	91.4	76.1	114	91.4	137	122	183

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		$1^1/_2$		$1^3/_4$		$1^1/_2$		$1^3/_4$		$1^1/_2$		$1^3/_4$	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	$1^1/_4$	145	218	154	230	130	194	138	207	141	212	150	224
	$1^3/_8$	148	222	156	234	132	198	140	210	144	216	152	228
	$1^1/_2$	150	225	158	238	134	202	143	214	146	219	154	232
	$1^5/_8$	153	229	161	241	137	205	145	218	149	223	157	235
	2	160	240	168	252	144	216	152	229	156	234	164	246
	3	180	269	188	282	164	246	172	258	176	263	184	275
Coped at Both Flanges	$1^1/_4$	137	205	137	205	122	183	122	183	137	205	137	205
	$1^3/_8$	141	212	141	212	127	190	127	190	141	212	141	212
	$1^1/_2$	146	219	146	219	132	197	132	197	146	219	146	219
	$1^5/_8$	151	227	151	227	137	205	137	205	149	223	151	227
	2	160	240	166	249	144	216	151	227	156	234	164	246
	3	180	269	188	282	164	246	172	258	176	263	184	275
Uncoped		305	457	305	457	283	424	283	424	305	457	305	457

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	609	914
OVS	566	848

Notes:
STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

* Tabulated values include $^1/_4$-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

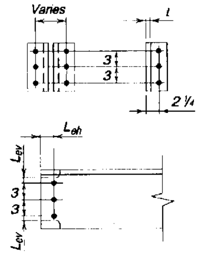
Beam F_y = 50 ksi
F_u = 65 ksi

Angle F_y = 36 ksi
F_u = 58 ksi

Table 10-1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

3 Rows W18, 16, 14, 12, 10⁺ ⁺Ltd. to W10x12, 15, 17, 19, 22, 26, 30	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD	1/2 ASD	1/2 LRFD
	A325/ F1852	N	—	44.6	66.9	55.7	83.6	66.9	100	89.2	134
		X	—	44.6	66.9	55.7	83.6	66.9	100	89.2	134
		SC Class A	STD	44.6	66.9	55.7	83.6	66.9	100	80.7	121
			OVS	39.7	59.5	49.6	74.4	58.3	87.4	58.3	87.4
			SSLT	44.6	66.9	55.7	83.6	66.9	100	68.6	103
		SC Class B	STD	44.6	66.9	55.7	83.6	66.9	100	89.2	134
			OVS	39.7	59.5	49.6	74.4	59.5	89.3	79.4	119
			SSLT	44.6	66.9	55.7	83.6	66.9	100	89.2	134
	A490	N	—	44.6	66.9	55.7	83.6	66.9	100	89.2	134
		X	—	44.6	66.9	55.7	83.6	66.9	100	89.2	134
		SC Class A	STD	44.6	66.9	55.7	83.6	66.9	100	89.2	134
			OVS	39.7	59.5	49.6	74.4	59.5	89.3	73.2	110
			SSLT	44.6	66.9	55.7	83.6	66.9	100	86.1	129
		SC Class B	STD	44.6	66.9	55.7	83.6	66.9	100	89.2	134
			OVS	39.7	59.5	49.6	74.4	59.5	89.3	79.4	119
			SSLT	44.6	66.9	55.7	83.6	66.9	100	89.2	134

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	109	163	117	176	96.7	145	105	157	105	157	113	169
	1 3/8	111	167	119	179	99.1	149	107	161	107	161	115	173
	1 1/2	114	171	122	183	102	152	110	165	110	165	118	177
	1 5/8	116	174	124	186	104	156	112	168	112	168	120	180
	2	124	185	132	197	111	167	119	179	119	179	128	191
	3	143	215	151	227	131	196	139	208	139	208	147	221
Coped at Both Flanges	1 1/4	99.9	150	99.9	150	89	133	89	133	99.9	150	99.9	150
	1 3/8	105	157	105	157	93.8	141	93.8	141	105	157	105	157
	1 1/2	110	165	110	165	98.7	148	98.7	148	110	165	110	165
	1 5/8	115	172	115	172	104	155	104	155	112	168	115	172
	2	124	185	129	194	111	167	118	177	119	179	128	191
	3	143	215	151	227	131	196	139	208	139	208	147	221
Uncoped		229	344	229	344	215	322	215	322	229	344	229	344

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	458	687
OVS	429	644

Notes:

STD = Standard holes	N = Threads included
OVS = Oversized holes	X = Threads excluded
SSLT = Short-slotted holes transverse to direction of load	SC = Slip critical

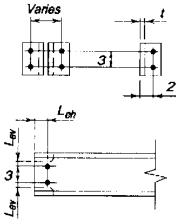
* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Beam	F_y = 50 ksi F_u = 65 ksi
Angle	F_y = 36 ksi F_u = 58 ksi

Table 10–1 (continued)
All-Bolted Double-Angle Connections

1-in. Bolts

Bolt and Angle Available Strength, kips

2 Rows	ASTM Desig.	Thread Cond.	Hole Type	Angle Thickness							
W12, 10, 8				1/4		5/16		3/8		1/2	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
	A325/ F1852	N	—	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
		X	—	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
		SC Class A	STD	28.3	42.4	35.3	53.0	42.4	63.6	53.8	80.7
			OVS	25.0	37.5	31.3	46.9	37.5	56.3	38.9	58.3
			SSLT	28.3	42.4	35.3	53.0	42.4	63.6	45.7	68.6
		SC Class B	STD	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
			OVS	25.0	37.5	31.3	46.9	37.5	56.3	50.0	75.0
			SSLT	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
	A490	N	—	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
		X	—	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
		SC Class A	STD	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
			OVS	25.0	37.5	31.3	46.9	37.5	56.3	48.8	73.2
			SSLT	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
		SC Class B	STD	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8
			OVS	25.0	37.5	31.3	46.9	37.5	56.3	50.0	75.0
			SSLT	28.3	42.4	35.3	53.0	42.4	63.6	56.6	84.8

Beam Web Available Strength per Inch Thickness, kips/in.

Hole Type		STD				OVS				SSLT			
		L_{eh}*											
L_{ev}, in.		1 1/2		1 3/4		1 1/2		1 3/4		1 1/2		1 3/4	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Coped at Top Flange Only	1 1/4	72.3	108	80.4	121	63.8	95.7	71.9	108	68.3	102	76.4	115
	1 3/8	74.8	112	82.9	124	66.2	99.3	74.3	112	70.7	106	78.8	118
	1 1/2	77.2	116	85.3	128	68.7	103	76.8	115	73.1	110	81.3	122
	1 5/8	79.6	119	87.8	132	71.1	107	79.2	119	75.6	113	83.7	126
	2	86.9	130	95.1	143	78.4	118	86.5	130	82.9	124	91.0	137
	3	106	160	115	172	97.9	147	106	159	102	154	111	166
Coped at Both Flanges	1 1/4	63.4	95.1	63.4	95.1	56.1	84.1	56.1	84.1	63.4	95.1	63.4	95.1
	1 3/8	68.3	102	68.3	102	60.9	91.4	60.9	91.4	68.3	102	68.3	102
	1 1/2	73.1	110	73.1	110	65.8	98.7	65.8	98.7	73.1	110	73.1	110
	1 5/8	78.0	117	78.0	117	70.7	106	70.7	106	75.6	113	78.0	117
	2	86.9	130	92.6	139	78.4	118	85.3	128	82.9	124	91.0	137
	3	106	160	115	172	97.9	147	106	159	102	154	111	166
Uncoped		154	230	154	230	146	219	146	219	154	230	154	230

Support Available Strength per Inch Thickness, kips/in.

Hole Type	ASD	LRFD
STD/ SSLT	307	461
OVS	293	439

Notes:

STD = Standard holes

OVS = Oversized holes

SSLT = Short-slotted holes transverse to direction of load

N = Threads included

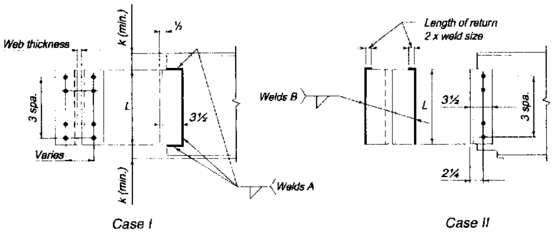
X = Threads excluded

SC = Slip critical

* Tabulated values include 1/4-in. reduction in end distance L_{eh} to account for possible underrun in beam length.

Table 10-2

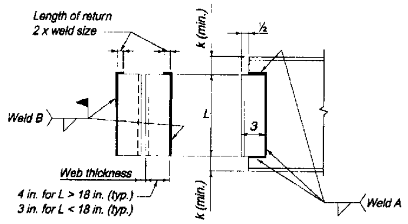
Bolted/Welded Double-Angle Connections



n	L	Welds A (70 ksi)				Welds B (70 ksi)			
		Weld Size, in.	R_n/Ω	ϕR_n	Minimum Web Thickness, in.	Weld Size, in.	R_n/Ω	ϕR_n	Minimum Support Thickness, in.
			kips	kips			kips	kips	
			ASD	LRFD			ASD	LRFD	
12	35½	5/16	**391**	586	0.476	3/8	**366**	550	0.286
		1/4	**313**	469	0.381	5/16	**305**	458	0.238
		3/16	**234**	352	0.286	1/4	**244**	366	0.190
11	32½	5/16	**366**	548	0.476	3/8	**331**	496	0.286
		1/4	**293**	439	0.381	5/16	**276**	414	0.238
		3/16	**219**	329	0.286	1/4	**221**	331	0.190
10	29½	5/16	**337**	506	0.476	3/8	**295**	443	0.286
		1/4	**270**	405	0.381	5/16	**246**	369	0.238
		3/16	**202**	303	0.286	1/4	**197**	295	0.190
9	26½	5/16	**309**	464	0.476	3/8	**259**	389	0.286
		1/4	**248**	371	0.381	5/16	**216**	324	0.238
		3/16	**186**	278	0.286	1/4	**173**	259	0.190
8	23½	5/16	**282**	422	0.476	3/8	**223**	335	0.286
		1/4	**225**	338	0.381	5/16	**186**	279	0.238
		3/16	**169**	253	0.286	1/4	**149**	223	0.190
7	20½	5/16	**253**	379	0.476	3/8	**187**	280	0.286
		1/4	**202**	304	0.381	5/16	**156**	234	0.238
		3/16	**152**	228	0.286	1/4	**125**	187	0.190
6	17½	5/16	**223**	334	0.476	3/8	**150**	226	0.286
		1/4	**178**	267	0.381	5/16	**125**	188	0.238
		3/16	**134**	200	0.286	1/4	**100**	150	0.190
5	14½	5/16	**191**	287	0.476	3/8	**115**	172	0.286
		1/4	**153**	229	0.381	5/16	**95.5**	143	0.238
		3/16	**115**	172	0.286	1/4	**76.4**	115	0.190
4	11½	5/16	**158**	237	0.476	3/8	**79.9**	120	0.286
		1/4	**126**	190	0.381	5/16	**66.6**	99.9	0.238
		3/16	**94.9**	142	0.286	1/4	**53.3**	79.9	0.190
3	8½	5/16	**122**	184	0.476	3/8	**48.1**	72.2	0.286
		1/4	**97.9**	147	0.381	5/16	**40.1**	60.2	0.238
		3/16	**73.4**	110	0.286	1/4	**32.1**	48.1	0.190
2	5½	5/16	**83.6**	125	0.476	3/8	**21.9**	32.8	0.286
		1/4	**66.9**	100	0.381	5/16	**18.2**	27.3	0.238
		3/16	**50.2**	75.3	0.286	1/4	**14.6**	21.9	0.190

ASD	LRFD		Beam	
$\Omega = 2.00$	$\phi = 0.75$		$F_y = 50$ ksi	$F_u = 65$ ksi

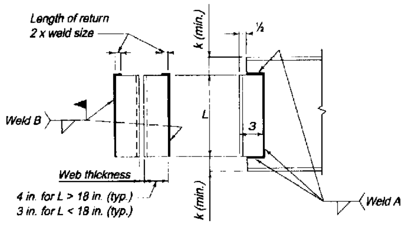
Table 10-3
All-Welded Double-Angle Connections



L	Welds A (70 ksi)				Welds B (70 ksi)			
	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.	Weld Size, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Minimum Web Thickness, in.
36	5/16	**395**	592	0.476	3/8	**372**	558	0.286
	1/4	**316**	474	0.381	5/16	**310**	465	0.238
	3/16	**237**	355	0.286	1/4	**248**	372	0.190
34	5/16	**378**	568	0.476	3/8	**349**	523	0.286
	1/4	**303**	454	0.381	5/16	**291**	436	0.238
	3/16	**227**	341	0.286	1/4	**232**	349	0.190
32	5/16	**361**	541	0.476	3/8	**325**	487	0.286
	1/4	**289**	433	0.381	5/16	**271**	406	0.238
	3/16	**217**	325	0.286	1/4	**217**	325	0.190
30	5/16	**342**	513	0.476	3/8	**301**	452	0.286
	1/4	**273**	410	0.381	5/16	**251**	377	0.238
	3/16	**205**	308	0.286	1/4	**201**	301	0.190
28	5/16	**323**	485	0.476	3/8	**277**	416	0.286
	1/4	**259**	388	0.381	5/16	**231**	347	0.238
	3/16	**194**	291	0.286	1/4	**185**	277	0.190
26	5/16	**305**	457	0.476	3/8	**253**	380	0.286
	1/4	**244**	366	0.381	5/16	**211**	317	0.238
	3/16	**183**	274	0.286	1/4	**169**	253	0.190
24	5/16	**286**	429	0.476	3/8	**229**	344	0.286
	1/4	**229**	343	0.381	5/16	**191**	286	0.238
	3/16	**172**	258	0.286	1/4	**153**	229	0.190
22	5/16	**267**	401	0.476	3/8	**205**	308	0.286
	1/4	**214**	321	0.381	5/16	**171**	256	0.238
	3/16	**160**	241	0.286	1/4	**137**	205	0.190
20	5/16	**248**	372	0.476	3/8	**181**	271	0.286
	1/4	**198**	298	0.381	5/16	**151**	226	0.238
	3/16	**149**	223	0.286	1/4	**121**	181	0.190
18	5/16	**228**	341	0.476	3/8	**157**	235	0.286
	1/4	**182**	273	0.381	5/16	**130**	196	0.238
	3/16	**137**	205	0.286	1/4	**104**	157	0.190
16	5/16	**207**	311	0.476	3/8	**148**	222	0.286
	1/4	**166**	249	0.381	5/16	**123**	185	0.238
	3/16	**124**	186	0.286	1/4	**98.5**	148	0.190

ASD	LRFD		Beam	
$\Omega = 2.00$	$\phi = 0.75$		$F_y = 50$ ksi	$F_u = 65$ ksi

Table 10–3 (continued)
All-Welded Double-Angle Connections



L	Welds A (70 ksi)				Welds B (70 ksi)			
	Weld Size, in.	R_n/Ω	ϕR_n	Minimum Web Thickness, in.	Weld Size, in.	R_n/Ω	ϕR_n	Minimum Web Thickness, in.
		kips	kips			kips	kips	
		ASD	LRFD			ASD	LRFD	
14	5/16	186	279	0.476	3/8	123	185	0.286
	1/4	149	223	0.381	5/16	103	154	0.238
	3/16	111	167	0.286	1/4	82.3	123	0.190
12	5/16	164	246	0.476	3/8	99.3	149	0.286
	1/4	131	197	0.381	5/16	82.8	124	0.238
	3/16	98.3	147	0.286	1/4	66.2	99.3	0.190
10	5/16	140	211	0.476	3/8	75.7	113	0.286
	1/4	112	169	0.381	5/16	63.1	94.6	0.238
	3/16	84.3	126	0.286	1/4	50.4	75.7	0.190
9	5/16	128	193	0.476	3/8	64.2	96.3	0.286
	1/4	103	154	0.381	5/16	53.5	80.2	0.238
	3/16	77.1	116	0.286	1/4	42.8	64.2	0.190
8	5/16	116	174	0.476	3/8	53.0	79.5	0.286
	1/4	92.9	139	0.381	5/16	44.2	66.3	0.238
	3/16	69.6	104	0.286	1/4	35.4	53.0	0.190
7	5/16	103	155	0.476	3/8	42.4	63.6	0.286
	1/4	82.5	124	0.381	5/16	35.3	53.0	0.238
	3/16	61.9	92.9	0.286	1/4	28.3	42.4	0.190
6	5/16	90.3	135	0.476	3/8	32.5	48.7	0.286
	1/4	72.3	108	0.381	5/16	27.0	40.6	0.238
	3/16	54.2	81.3	0.286	1/4	21.6	32.5	0.190
5	5/16	77.1	116	0.476	3/8	23.4	35.1	0.286
	1/4	61.7	92.6	0.381	5/16	19.5	29.2	0.238
	3/16	46.3	69.4	0.286	1/4	15.6	23.4	0.190
4	5/16	64.2	96.3	0.476	3/8	15.5	23.2	0.286
	1/4	51.3	77.0	0.381	5/16	12.9	19.3	0.238
	3/16	38.5	57.8	0.286	1/4	10.3	15.5	0.190

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

Beam	
$F_y = 50$ ksi	$F_u = 65$ ksi

SHEAR END-PLATE CONNECTIONS

A shear end-plate connection is made with a plate length less than the supported beam depth, as illustrated in Figure 10–6. The end plate is always shop-welded to the beam web with fillet welds on each side and usually field-bolted to the supporting member. Welds connecting the end plate to the beam web should not be returned across the thickness of the beam web at the top or bottom of the end plate because of the danger of creating a notch in the beam web.

If the end plate is field-welded to the support, adequate flexibility must be provided in the connection. Line welds are placed along the vertical edges of the plate with a return at the top per AISC Specification Section J2.2b. Note that welding across the entire top of the plate must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for simple shear connections.

Design Checks

The available strength of a shear end-plate connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Note that the limit-state of shear rupture of the beam web must be checked along the length of weld connecting the end plate to the beam web. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Recommended End-Plate Dimensions and Thickness

To provide for stability during erection, it is recommended that the minimum end-plate length be one-half the T-dimension of the beam to be supported. The maximum length of the end plate must be compatible with the clear distance between the flanges of an uncoped beam and the remaining clear distance of a coped beam.

To provide for flexibility, the combination of plate thickness and gage should be consistent with the recommendations given previously for a double-angle connection of similar thickness and gage.

Shop and Field Practices

When framing to a column web, the associated constructability considerations should be addressed (see the preceding discussion under "Constructability Considerations").

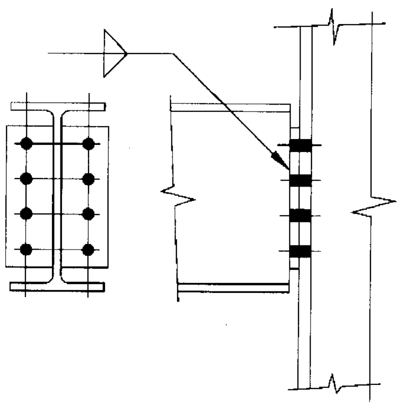


Figure 10–6. Shear end-plate connections.

When framing to a column flange, provision must be made for possible mill variation in the depth of the columns, particularly in fairly long runs (i.e., six or more bays of framing). The beam length can be shortened to provide for mill overrun with shims furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun. Shear end-plate connections require close control in cutting the beam to the proper length and in squaring the beam ends such that both end plates are parallel, particularly when beams are cambered.

Table 10–4. Bolted/Welded Shear End-Plate Connections

Table 10–4 is a design aid for shear end-plate connections bolted to the supporting member and welded to the supported beam. Available strengths are tabulated for supported and supporting member material with F_y = 50 ksi and F_u = 65 ksi and end-plate material with F_y = 36 ksi and F_u = 58 ksi. Electrode strength is assumed to be 70 ksi. All values, including slip-critical bolt available strengths, are for comparison with the LRFD load combination for LRFD design and the ASD load combination for ASD design.

Tabulated bolt and end-plate available strengths consider the limit-states of bolt shear, bolt bearing on the end plate, shear yielding of the end plate, shear rupture of the end plate, and block shear rupture of the end plate. Values are included for 2 through 12 rows of $^3/_4$-in., $^7/_8$-in., and 1-in. diameter ASTM A325, F1852 and A490 bolts at 3-in. spacing. End-plate edge distances L_{ev} and L_{eh} are assumed to be $1^1/_4$ in.

Tabulated weld available strengths consider the limit-state of weld shear assuming an effective weld length equal to the end-plate length minus twice the weld size. The tabulated minimum beam web thickness matches the shear rupture strength of the web material to the strength of the weld metal. As derived in Part 9, the minimum supported beam web thickness for two lines of weld is

$$t_{min} = \frac{6.19D}{F_u}$$

where D is the number of sixteenths-of-an-inch in the weld size. When less than the minimum material thickness is present, the tabulated weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness.

Tabulated supporting member available strengths, per in. of flange or web thickness, consider the limit-state of bolt bearing.

W44

Table 10–4
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts 12 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	197	295	246	369	254	382
	X	—	197	295	246	369	295	443
	SC Class A	STD	177	266	177	266	177	266
		OVS	128	192	128	192	128	192
		SSLT	151	226	151	226	151	226
	SC Class B	STD	197	295	246	369	253	380
		OVS	183	274	183	274	183	274
		SSLT	195	293	215	323	215	323
A490	N	—	197	295	246	369	295	443
	X	—	197	295	246	369	295	443
	SC Class A	STD	197	295	221	332	221	332
		OVS	160	240	160	240	160	240
		SSLT	188	282	188	282	188	282
	SC Class B	STD	197	295	246	369	295	443
		OVS	196	294	229	343	229	343
		SSLT	195	293	244	366	269	403

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
$^3/_{16}$	0.286	196	293	1400	2110
$^1/_4$	0.381	260	390		
$^5/_{16}$	0.476	324	486		
$^3/_8$	0.571	387	581		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

$^3/_4$-in.
Bolts
11 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	181	271	226	338	233	350
	X	—	181	271	226	338	271	406
	SC Class A	STD	162	244	162	244	162	244
		OVS	117	176	117	176	117	176
		SSLT	138	207	138	207	138	207
	SC Class B	STD	181	271	226	338	232	348
		OVS	168	251	168	251	168	251
		SSLT	179	269	197	296	197	296
A490	N	—	181	271	226	338	271	406
	X	—	181	271	226	338	271	406
	SC Class A	STD	181	271	203	305	203	305
		OVS	147	220	147	220	147	220
		SSLT	173	259	173	259	173	259
	SC Class B	STD	181	271	226	338	271	406
		OVS	180	269	210	314	210	314
		SSLT	179	269	224	336	247	370

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
$^3/_{16}$	0.286	179	268	1290	1930
$^1/_4$	0.381	238	356		
$^5/_{16}$	0.476	296	444		
$^3/_8$	0.571	354	530		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44, 40, 36

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

$^3/_4$-in. Bolts 10 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
A325/F1852	N	—	164	246	205	308	212	318
	X	—	164	246	205	308	246	370
	SC Class A	STD	148	221	148	221	148	221
		OVS	107	160	107	160	107	160
		SSLT	126	188	126	188	126	188
	SC Class B	STD	164	246	205	308	211	316
		OVS	152	229	152	229	152	229
		SSLT	163	244	179	269	179	269
A490	N	—	164	246	205	308	246	370
	X	—	164	246	205	308	246	370
	SC Class A	STD	164	246	185	277	185	277
		OVS	133	200	133	200	133	200
		SSLT	157	235	157	235	157	235
	SC Class B	STD	164	246	205	308	246	370
		OVS	163	245	190	286	190	286
		SSLT	163	244	204	306	224	336

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	ASD	LRFD
$^3/_{16}$	0.286	162	243	1170	1760
$^1/_4$	0.381	215	323		
$^5/_{16}$	0.476	268	402		
$^3/_8$	0.571	320	480		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

3/4-in.
Bolts
9 Rows

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	148	222	185	278	191	286
	X	—	148	222	185	278	222	333
	SC Class A	STD	133	199	133	199	133	199
		OVS	96.0	144	96.0	144	96.0	144
		SSLT	113	169	113	169	113	169
	SC Class B	STD	148	222	185	278	190	285
		OVS	137	206	137	206	137	206
		SSLT	147	220	161	242	161	242
A490	N	—	148	222	185	278	222	333
	X	—	148	222	185	278	222	333
	SC Class A	STD	148	222	166	249	166	249
		OVS	120	180	120	180	120	180
		SSLT	141	212	141	212	141	212
	SC Class B	STD	148	222	185	278	222	333
		OVS	147	221	171	257	171	257
		SSLT	147	220	183	275	202	303

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	145	218	1050	1580
1/4	0.381	193	290		
5/16	0.476	240	360		
3/8	0.571	287	430		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44, 40,
36, 33,
30

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts
8 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	132	198	165	247	170	254
	X	—	132	198	165	247	198	297
	SC Class A	STD	118	177	118	177	118	177
		OVS	85.3	128	85.3	128	85.3	128
		SSLT	100	151	100	151	100	151
	SC Class B	STD	132	198	165	247	169	253
		OVS	122	183	122	183	122	183
		SSLT	131	196	143	215	143	215
A490	N	—	132	198	165	247	198	297
	X	—	132	198	165	247	198	297
	SC Class A	STD	132	198	148	221	148	221
		OVS	107	160	107	160	107	160
		SSLT	126	188	126	188	126	188
	SC Class B	STD	132	198	165	247	198	297
		OVS	131	197	152	229	152	229
		SSLT	131	196	163	245	179	269

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	129	193	936	1400
1/4	0.381	171	256		
5/16	0.476	212	318		
3/8	0.571	253	380		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

3/4-in. Bolts
7 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30, 27, 24

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	116	174	145	217	148	223
	X	—	116	174	145	217	174	260
	SC Class A	STD	103	155	103	155	103	155
		OVS	74.7	112	74.7	112	74.7	112
		SSLT	87.9	132	87.9	132	87.9	132
	SC Class B	STD	116	174	145	217	148	221
		OVS	107	160	107	160	107	160
		SSLT	114	172	126	188	126	188
A490	N	—	116	174	145	217	174	260
	X	—	116	174	145	217	174	260
	SC Class A	STD	116	174	129	194	129	194
		OVS	93.3	140	93.3	140	93.3	140
		SSLT	110	165	110	165	110	165
	SC Class B	STD	116	174	145	217	174	260
		OVS	115	172	133	200	133	200
		SSLT	114	172	143	214	157	235

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	112	168	819	1230
1/4	0.381	148	223		
5/16	0.476	184	277		
3/8	0.571	220	330		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44, 40,
36, 33,
30, 27,
24, 21

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in.
Bolts
6 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	99.5	149	124	187	127	191
	X	—	99.5	149	124	187	149	224
	SC Class A	STD	88.6	133	88.6	133	88.6	133
		OVS	64.0	96.0	64.0	96.0	64.0	96.0
		SSLT	75.3	113	75.3	113	75.3	113
	SC Class B	STD	99.5	149	124	187	127	190
		OVS	91.4	137	91.4	137	91.4	137
		SSLT	98.2	147	108	161	108	161
A490	N	—	99.5	149	124	187	149	224
	X	—	99.5	149	124	187	149	224
	SC Class A	STD	99.5	149	111	166	111	166
		OVS	80.0	120	80.0	120	80.0	120
		SSLT	94.1	141	94.1	141	94.1	141
	SC Class B	STD	99.5	149	124	187	149	224
		OVS	98.6	148	114	171	114	171
		SSLT	98.2	147	123	184	134	202

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	95.4	143	702	1050
1/4	0.381	126	189		
5/16	0.476	157	235		
3/8	0.571	187	280		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

3/4-in.
Bolts
5 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W30, 27,
24, 21,
18

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	83.3	125	104	156	106	159
	X	—	83.3	125	104	156	125	187
	SC Class A	STD	73.8	111	73.8	111	73.8	111
		OVS	53.3	80.0	53.3	80.0	53.3	80.0
		SSLT	62.8	94.1	62.8	94.1	62.8	94.1
	SC Class B	STD	83.3	125	104	156	105	158
		OVS	76.2	114	76.2	114	76.2	114
		SSLT	82	123	89.6	134	89.6	134
A490	N	—	83.3	125	104	156	125	187
	X	—	83.3	125	104	156	125	187
	SC Class A	STD	83.3	125	92.3	138	92.3	138
		OVS	66.7	100	66.7	100	66.7	100
		SSLT	78.4	118	78.4	118	78.4	118
	SC Class B	STD	83.3	125	104	156	125	187
		OVS	82.4	124	95.2	143	95.2	143
		SSLT	82	123	102	154	112	168

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
$^3/_{16}$	0.286	78.7	118	585	878
$^1/_4$	0.381	104	156		
$^5/_{16}$	0.476	129	193		
$^3/_8$	0.571	153	230		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W24, 21, 18, 16

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

$3/4$-in. Bolts
4 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. $1/4$ ASD	$1/4$ LRFD	$5/16$ ASD	$5/16$ LRFD	$3/8$ ASD	$3/8$ LRFD
A325/F1852	N	—	67.1	101	83.9	126	84.8	127
	X	—	67.1	101	83.9	126	101	151
	SC Class A	STD	59.1	88.6	59.1	88.6	59.1	88.6
		OVS	42.7	64.0	42.7	64.0	42.7	64.0
		SSLT	50.2	75.3	50.2	75.3	50.2	75.3
	SC Class B	STD	67.1	101	83.9	126	84.4	127
		OVS	61.0	91.4	61.0	91.4	61.0	91.4
		SSLT	65.8	98.7	71.7	108	71.7	108
A490	N	—	67.1	101	83.9	126	101	151
	X	—	67.1	101	83.9	126	101	151
	SC Class A	STD	67.1	101	73.8	111	73.8	111
		OVS	53.3	80.0	53.3	80.0	53.3	80.0
		SSLT	62.8	94.1	62.8	94.1	62.8	94.1
	SC Class B	STD	67.1	101	83.9	126	101	151
		OVS	65.3	97.9	76.2	114	76.2	114
		SSLT	65.8	98.7	82.2	123	89.6	134

Weld and Beam Web Available Strength, kips / **Support Available Strength per Inch Thickness, kips/in.**

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Support ASD	Support LRFD
$3/16$	0.286	61.9	92.9	468	702
$1/4$	0.381	81.7	123		
$5/16$	0.476	101	151		
$3/8$	0.571	120	180		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

$^3/_4$-in.
Bolts
3 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W18, 16, 14, 12, 10*

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
A325/F1852	N	—	50.9	76.4	63.6	95.4	63.6	95.4
	X	—	50.9	76.4	63.7	95.5	76.4	115.0
	SC Class A	STD	44.3	66.4	44.3	66.4	44.3	66.4
		OVS	32.0	48.0	32.0	48.0	32.0	48.0
		SSLT	37.7	56.5	37.7	56.5	37.7	56.5
	SC Class B	STD	50.9	76.4	63.3	94.9	63.3	94.9
		OVS	45.7	68.6	45.7	68.6	45.7	68.6
		SSLT	49.6	74.4	53.8	80.7	53.8	80.7
A490	N	—	50.9	76.4	63.7	95.5	76.4	115.0
	X	—	50.9	76.4	63.7	95.5	76.4	115.0
	SC Class A	STD	50.9	76.4	55.4	83.1	55.4	83.1
		OVS	40.0	60.0	40.0	60.0	40.0	60.0
		SSLT	47.1	70.6	47.1	70.6	47.1	70.6
	SC Class B	STD	50.9	76.4	63.7	95.5	76.4	115.0
		OVS	47.9	71.8	57.1	85.7	57.1	85.7
		SSLT	49.8	74.4	62.0	92.9	67.2	101.0

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	ASD	LRFD
$^3/_{16}$	0.286	45.2	67.9	351	526
$^1/_4$	0.381	59.4	89.1		
$^5/_{16}$	0.476	73.1	110		
$^3/_8$	0.571	88.3	129		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical
*Limited to W10×12, 15,17, 19, 22, 26, 30

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W12, 10, 8

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

3/4-in. Bolts 2 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	32.6	48.9	40.8	61.2	42.4	63.6
	X	—	32.6	48.9	40.8	61.2	48.9	73.4
	SC Class A	STD	29.5	44.3	29.5	44.3	29.5	44.3
		OVS	21.3	32.0	21.3	32.0	21.3	32.0
		SSLT	25.1	37.7	25.1	37.7	25.1	37.7
	SC Class B	STD	32.6	48.9	40.8	61.2	42.2	63.3
		OVS	30.5	45.7	30.5	45.7	30.5	45.7
		SSLT	32.6	48.9	35.9	53.8	35.9	53.8
A490	N	—	32.6	48.9	40.8	61.2	48.9	73.4
	X	—	32.6	48.9	40.8	61.2	48.9	73.4
	SC Class A	STD	32.6	48.9	36.9	55.4	36.9	55.4
		OVS	26.7	40.0	26.7	40.0	26.7	40.0
		SSLT	31.4	47.1	31.4	47.1	31.4	47.1
	SC Class B	STD	32.6	48.9	40.8	61.2	48.9	73.4
		OVS	30.5	45.7	38.1	57.1	38.1	57.1
		SSLT	32.6	48.9	40.8	61.2	44.8	67.2

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	28.5	42.8	234	351
1/4	0.381	37.1	55.7		
5/16	0.476	45.2	67.9		
3/8	0.571	52.9	79.4		

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

7/8-in.
Bolts
12 Rows

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W44

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
A325/F1852	N	—	196	294	245	367	294	441
	X	—	196	294	245	367	294	441
	SC Class A	STD	196	294	245	367	247	370
		OVS	178	267	178	267	178	267
		SSLT	194	292	210	315	210	315
	SC Class B	STD	196	294	245	367	294	441
		OVS	191	287	239	359	255	382
		SSLT	194	292	243	365	292	438
A490	N	—	196	294	245	367	294	441
	X	—	196	294	245	367	294	441
	SC Class A	STD	196	294	245	367	294	441
		OVS	191	287	224	336	224	336
		SSLT	194	292	243	365	264	395
	SC Class B	STD	196	294	245	367	294	441
		OVS	191	287	239	359	287	431
		SSLT	194	292	243	365	292	438

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	ASD	LRFD
3/16	0.286	196	293	1640	2460
1/4	0.381	260	390		
5/16	0.476	324	486		
3/8	0.571	387	581		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

W44, 40

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts 11 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	180	269	225	337	269	404
	X	—	180	269	225	337	269	404
	SC Class A	STD	180	269	225	337	226	339
		OVS	163	245	163	245	163	245
		SSLT	178	267	192	288	192	288
	SC Class B	STD	180	269	225	337	269	404
		OVS	175	263	219	328	233	350
		SSLT	178	267	223	334	267	401
A490	N	—	180	269	225	337	269	404
	X	—	180	269	225	337	269	404
	SC Class A	STD	180	269	225	337	269	404
		OVS	175	263	205	308	205	308
		SSLT	178	267	223	334	242	362
	SC Class B	STD	180	269	225	337	269	404
		OVS	175	263	219	328	263	394
		SSLT	178	267	223	334	267	401

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
$^3/_{16}$	0.286	179	268
$^1/_4$	0.381	238	356
$^5/_{16}$	0.476	296	444
$^3/_8$	0.571	354	530

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
1500	2250

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

7/8-in.
Bolts
10 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	163	245	204	306	245	368
	X	—	163	245	204	306	245	368
	SC Class A	STD	163	245	204	306	245	308
		OVS	149	223	149	223	149	223
		SSLT	162	243	175	262	175	262
	SC Class B	STD	163	245	204	306	245	368
		OVS	159	238	198	298	212	318
		SSLT	162	243	203	304	243	365
A490	N	—	163	245	204	306	245	368
	X	—	163	245	204	306	245	368
	SC Class A	STD	163	245	204	306	245	368
		OVS	159	238	187	280	187	280
		SSLT	162	243	203	304	220	329
	SC Class B	STD	163	245	204	306	245	368
		OVS	159	238	198	298	238	357
		SSLT	162	243	203	304	243	365

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	162	243	1370	2050
1/4	0.381	215	323		
5/16	0.476	268	402		
3/8	0.571	320	480		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44, 40, 36, 33

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
9 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	147	221	184	276	221	331
	X	—	147	221	184	276	221	331
	SC Class A	STD	147	221	184	276	185	278
		OVS	134	201	134	201	134	201
		SSLT	146	219	157	236	157	236
	SC Class B	STD	147	221	184	276	221	331
		OVS	142	214	178	267	191	287
		SSLT	146	219	182	273	219	328
A490	N	—	147	221	184	276	221	331
	X	—	147	221	184	276	221	331
	SC Class A	STD	147	221	184	276	221	331
		OVS	142	214	168	252	168	252
		SSLT	146	219	182	273	198	297
	SC Class B	STD	147	221	184	276	221	331
		OVS	142	214	178	267	214	321
		SSLT	146	219	182	273	219	328

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
$^3/_{16}$	0.286	145	218	1230	1840
$^1/_4$	0.381	193	290		
$^5/_{16}$	0.476	240	360		
$^3/_8$	0.571	287	430		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

$^{7}/_{8}$-in. Bolts 8 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. $^{1}/_{4}$ ASD	$^{1}/_{4}$ LRFD	$^{5}/_{16}$ ASD	$^{5}/_{16}$ LRFD	$^{3}/_{8}$ ASD	$^{3}/_{8}$ LRFD
A325/F1852	N	—	131	197	164	246	197	295
	X	—	131	197	164	246	197	295
	SC Class A	STD	131	197	164	246	165	247
		OVS	119	178	119	178	119	178
		SSLT	130	194	140	210	140	210
	SC Class B	STD	131	197	164	246	197	295
		OVS	126	189	158	237	170	255
		SSLT	130	194	162	243	194	292
A490	N	—	131	197	164	246	197	295
	X	—	131	197	164	246	197	295
	SC Class A	STD	131	197	164	246	197	295
		OVS	126	189	149	224	149	224
		SSLT	130	194	162	243	176	264
	SC Class B	STD	131	197	164	246	197	295
		OVS	126	189	158	237	189	284
		SSLT	130	194	162	243	194	292

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	Support Available Strength per Inch Thickness, kips/in. ASD	Support Available Strength per Inch Thickness, kips/in. LRFD
$^{3}/_{16}$	0.286	129	193	1090	1640
$^{1}/_{4}$	0.381	171	256		
$^{5}/_{16}$	0.476	212	318		
$^{3}/_{8}$	0.571	253	380		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W44, 40, 36, 33, 30, 27, 24

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts
7 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
A325/F1852	N	—	115	172	144	215	172	258
	X	—	115	172	144	215	172	258
	SC Class A	STD	115	172	144	215	144	216
		OVS	104	156	104	156	104	156
		SSLT	113	170	122	184	122	184
	SC Class B	STD	115	172	144	215	172	258
		OVS	110	165	137	206	149	223
		SSLT	113	170	142	213	170	255
A490	N	—	115	172	144	215	172	258
	X	—	115	172	144	215	172	258
	SC Class A	STD	115	172	144	215	172	258
		OVS	110	165	131	196	131	196
		SSLT	113	170	142	213	154	231
	SC Class B	STD	115	172	144	215	172	258
		OVS	110	165	137	206	165	247
		SSLT	113	170	142	213	170	255

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
3/16	0.286	112	168
1/4	0.381	148	223
5/16	0.476	184	277
3/8	0.571	220	330

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
956	1430

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

7/8-in.
Bolts
6 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W40, 36, 33, 30, 27, 24, 21

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
A325/F1852	N	—	98.6	148	123	185	148	222
	X	—	98.6	148	123	185	148	222
	SC Class A	STD	98.6	148	123	185	123	185
		OVS	89.2	134	89.2	134	89.2	134
		SSLT	97.3	146	105	157	105	157
	SC Class B	STD	98.6	148	123	185	148	222
		OVS	93.5	140	117	175	127	191
		SSLT	97.3	146	122	182	146	219
A490	N	—	98.6	148	123	185	148	222
	X	—	98.6	148	123	185	148	222
	SC Class A	STD	98.6	148	123	185	148	222
		OVS	93.5	140	112	168	112	168
		SSLT	97.3	146	122	182	132	198
	SC Class B	STD	98.6	148	123	185	148	222
		OVS	93.5	140	117	175	140	210
		SSLT	97.3	146	122	182	146	219

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD
$^3/_{16}$	0.286	95.4	143
$^1/_4$	0.381	126	189
$^5/_{16}$	0.476	157	235
$^3/_8$	0.571	187	280

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
819	1230

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

**W30, 27,
24, 21,
18**

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

**7/8-in.
Bolts
5 Rows**

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	82.4	124	103	155	124	185
	X	—	82.4	124	103	155	124	185
	SC Class A	STD	82.4	124	103	154	103	154
		OVS	74.3	111	74.3	111	74.3	111
		SSLT	81.1	122	87.4	131	87.4	131
	SC Class B	STD	82.4	124	103	155	124	185
		OVS	77.2	116	96.5	145	106	159
		SSLT	81.1	122	101	152	122	182
A490	N	—	82.4	124	103	155	124	185
	X	—	82.4	124	103	155	124	185
	SC Class A	STD	82.4	124	103	155	124	185
		OVS	77.2	116	93.3	140	93.3	140
		SSLT	81.1	122	101	152	110	165
	SC Class B	STD	82.4	124	103	155	124	185
		OVS	77.2	116	96.5	145	116	174
		SSLT	81.1	122	101	152	122	182

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	78.7	118
1/4	0.381	104	156
5/16	0.476	193	193
3/8	0.571	153	230

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
683	1020

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

7/8-in.
Bolts
4 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W24, 21, 18, 16

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. 1/4 ASD	1/4 LRFD	5/16 ASD	5/16 LRFD	3/8 ASD	3/8 LRFD
A325/F1852	N	—	65.3	97.9	81.6	122	97.9	147
	X	—	65.3	97.9	81.6	122	97.9	147
	SC Class A	STD	65.3	97.9	81.6	122	82.3	123
		OVS	59.4	89.2	59.4	89.2	59.4	89.2
		SSLT	64.9	97.3	69.9	105	69.9	105
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147
		OVS	60.9	91.4	76.1	114	84.9	127
		SSLT	64.9	97.3	81.1	122	97.3	146
A490	N	—	65.3	97.9	81.6	122	97.9	147
	X	—	65.3	97.9	81.6	122	97.9	147
	SC Class A	STD	65.3	97.9	81.6	122	97.9	147
		OVS	60.9	91.4	74.7	112	74.7	112
		SSLT	64.9	97.3	81.1	122	87.9	132
	SC Class B	STD	65.3	97.9	81.6	122	97.9	147
		OVS	60.9	91.4	76.1	114	91.4	137
		SSLT	64.9	97.3	81.1	122	97.3	146

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	ASD	LRFD
$^3/_{16}$	0.286	61.9	92.9	546	819
$^1/_4$	0.381	81.7	123		
$^5/_{16}$	0.476	101	151		
$^3/_8$	0.571	120	180		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W18, 16, 14, 12, 10*

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

7/8-in. Bolts 3 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	47.9	71.8	59.8	89.7	71.8	108
	X	—	47.9	71.8	59.8	89.7	71.8	108
	SC Class A	STD	47.9	71.8	59.8	89.7	61.7	92.5
		OVS	44.6	66.9	44.6	66.9	44.6	66.9
		SSLT	47.9	71.8	52.4	78.7	52.4	78.7
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108
		OVS	44.6	66.9	55.7	83.6	63.7	95.5
		SSLT	47.9	71.8	59.8	89.7	71.8	108
A490	N	—	47.9	71.8	59.8	89.7	71.8	108
	X	—	47.9	71.8	59.8	89.7	71.8	108
	SC Class A	STD	47.9	71.8	59.8	89.7	71.8	108
		OVS	44.6	66.9	55.7	83.6	56.0	84.0
		SSLT	47.9	71.8	59.8	89.7	65.9	98.8
	SC Class B	STD	47.9	71.8	59.8	89.7	71.8	108
		OVS	44.6	66.9	55.7	83.6	66.9	100
		SSLT	47.9	71.8	59.8	89.7	71.8	108

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
3/16	0.286	45.2	67.9	409	614
1/4	0.381	59.4	89.1		
5/16	0.476	73.1	110		
3/8	0.571	86.3	129		

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical
*Limited to W10×12, 15, 17, 19, 22, 26, 30

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

$^7/_8$-in.
Bolts
2 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W12, 10, 8

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in. $^1/_4$ ASD	$^1/_4$ LRFD	$^5/_{16}$ ASD	$^5/_{16}$ LRFD	$^3/_8$ ASD	$^3/_8$ LRFD
A325/F1852	N	—	30.5	45.7	38.1	57.1	45.7	68.5
	X	—	30.5	45.7	38.1	57.1	45.7	68.5
	SC Class A	STD	30.5	45.7	38.1	57.1	41.1	61.7
		OVS	28.3	42.4	29.7	44.6	29.7	44.6
		SSLT	30.5	45.7	35.0	52.4	35.0	52.4
	SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5
		OVS	28.3	42.4	35.3	53.0	42.4	63.6
		SSLT	30.5	45.7	38.1	57.1	45.7	68.5
A490	N	—	30.5	45.7	38.1	57.1	45.7	68.5
	X	—	30.5	45.7	38.1	57.1	45.7	68.5
	SC Class A	STD	30.5	45.7	38.1	57.1	45.7	68.5
		OVS	28.3	42.4	35.3	53.0	37.3	56.0
		SSLT	30.5	45.7	38.1	57.1	43.9	65.9
	SC Class B	STD	30.5	45.7	38.1	57.1	45.7	68.5
		OVS	28.3	42.4	35.3	53.0	42.4	63.6
		SSLT	30.5	45.7	38.1	57.1	45.7	68.5

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips ASD	ϕR_n kips LRFD	ASD	LRFD
$^3/_{16}$	0.286	28.5	42.8	273	409
$^1/_4$	0.381	37.1	55.7		
$^5/_{16}$	0.476	45.2	67.9		
$^3/_8$	0.571	52.9	79.4		

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

W44

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
12 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	191	287	239	359	287	431
	X	—	191	287	239	359	287	431
	SC Class A	STD	191	287	239	359	287	431
		OVS	172	258	215	322	233	350
		SSLT	191	287	239	359	274	411
	SC Class B	STD	191	287	239	359	287	431
		OVS	172	258	215	322	258	387
		SSLT	191	287	239	359	287	431
A490	N	—	191	287	239	359	287	431
	X	—	191	287	239	359	287	431
	SC Class A	STD	191	287	239	359	287	431
		OVS	172	258	215	322	258	387
		SSLT	191	287	239	359	287	431
	SC Class B	STD	191	287	239	359	287	431
		OVS	172	258	215	322	258	387
		SSLT	191	287	239	359	287	431

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
$^3/_{16}$	0.286	196	293
$^1/_4$	0.381	260	390
$^5/_{16}$	0.476	324	486
$^3/_8$	0.571	387	581

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
1820 STD/SSLT	2730 STD/SSLT
1660 OVS	2490 OVS

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

1-in.
Bolts
11 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^{1}/_{4}$		$^{5}/_{16}$		$^{3}/_{8}$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	175	263	219	328	263	394
	X	—	175	263	219	328	263	394
	SC Class A	STD	175	263	219	328	263	394
		OVS	157	236	196	295	214	321
		SSLT	175	263	219	328	251	377
	SC Class B	STD	175	263	219	328	263	394
		OVS	157	236	196	295	236	354
		SSLT	175	263	219	328	263	394
A490	N	—	175	263	219	328	263	394
	X	—	175	263	219	328	263	394
	SC Class A	STD	175	263	219	328	263	394
		OVS	157	236	196	295	236	354
		SSLT	175	263	219	328	263	394
	SC Class B	STD	175	263	219	328	263	394
		OVS	157	236	196	295	236	354
		SSLT	175	263	219	328	263	394

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
$^{3}/_{16}$	0.286	179	268
$^{1}/_{4}$	0.381	238	356
$^{5}/_{16}$	0.476	296	444
$^{3}/_{8}$	0.571	354	530

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
1670 STD/ SSLT	2500 STD/ SSLT
1520 OVS	2280 OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

W44, 40, 36

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
10 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	159	238	198	298	238	357
	X	—	159	238	198	298	238	357
	SC Class A	STD	159	238	198	298	238	357
		OVS	142	214	178	267	194	291
		SSLT	159	238	198	298	229	343
	SC Class B	STD	159	238	198	298	238	357
		OVS	142	214	178	267	214	321
		SSLT	159	238	198	298	238	357
A490	N	—	159	238	198	298	238	357
	X	—	159	238	198	298	238	357
	SC Class A	STD	159	238	198	298	238	357
		OVS	142	214	178	267	214	321
		SSLT	159	238	198	298	238	357
	SC Class B	STD	159	238	198	298	238	357
		OVS	142	214	178	267	214	321
		SSLT	159	238	198	298	238	357

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	162	243
1/4	0.381	215	323
5/16	0.476	268	402
3/8	0.571	320	480

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
1520	STD/ SSLT	2270	STD/ SSLT
1380	OVS	2080	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

1-in.
Bolts
9 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	142	214	178	267	214	321
	X	—	142	214	178	267	214	321
	SC Class A	STD	142	214	178	267	214	321
		OVS	128	192	160	240	175	262
		SSLT	142	214	178	267	206	309
	SC Class B	STD	142	214	178	267	214	321
		OVS	128	192	160	240	192	288
		SSLT	142	214	178	267	214	321
A490	N	—	142	214	178	267	214	321
	X	—	142	214	178	267	214	321
	SC Class A	STD	142	214	178	267	214	321
		OVS	128	192	160	240	192	288
		SSLT	142	214	178	267	214	321
	SC Class B	STD	142	214	178	267	214	321
		OVS	128	192	160	240	192	288
		SSLT	142	214	178	267	214	321

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω	ϕR_n
		kips	kips
		ASD	LRFD
$^3/_{16}$	0.286	145	218
$^1/_4$	0.381	193	290
$^5/_{16}$	0.476	240	360
$^3/_8$	0.571	287	430

Support Available Strength per Inch Thickness, kips/in.

ASD	LRFD
1370 STD/SSLT	2050 STD/SSLT
1250 OVS	1870 OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

W44, 40, 36, 33, 30

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
8 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	126	189	158	237	189	284
	X	—	126	189	158	237	189	284
	SC Class A	STD	126	189	158	237	189	284
		OVS	113	170	141	212	155	233
		SSLT	126	189	158	237	183	274
	SC Class B	STD	126	189	158	237	189	284
		OVS	113	170	141	212	170	254
		SSLT	126	189	158	237	189	284
A490	N	—	126	189	158	237	189	284
	X	—	126	189	158	237	189	284
	SC Class A	STD	126	189	158	237	189	284
		OVS	113	170	141	212	170	254
		SSLT	126	189	158	237	189	284
	SC Class B	STD	126	189	158	237	189	284
		OVS	113	170	141	212	170	254
		SSLT	126	189	158	237	189	284

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
$^3/_{16}$	0.286	129	193
$^1/_4$	0.381	171	256
$^5/_{16}$	0.476	212	318
$^3/_8$	0.571	253	380

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
1210	STD/ SSLT	1820	STD/ SSLT
1110	OVS	1670	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

1-in.
Bolts
7 Rows

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

W44, 40, 36, 33, 30, 27, 24

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	110	165	137	206	165	247
	X	—	110	165	137	206	165	247
	SC Class A	STD	110	165	137	206	165	247
		OVS	98.4	148	123	185	136	204
		SSLT	110	165	137	206	160	240
	SC Class B	STD	110	165	137	206	165	247
		OVS	98.4	148	123	185	148	221
		SSLT	110	165	137	206	165	247
A490	N	—	110	165	137	206	165	247
	X	—	110	165	137	206	165	247
	SC Class A	STD	110	165	137	206	165	247
		OVS	98.4	148	123	185	148	221
		SSLT	110	165	137	206	165	247
	SC Class B	STD	110	165	137	206	165	247
		OVS	98.4	148	123	185	148	221
		SSLT	110	165	137	206	165	247

Weld and Beam Web Available Strength, kips				Support Available Strength per Inch Thickness, kips/in.	
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips		
		ASD	LRFD	ASD	LRFD
$^3/_{16}$	0.286	112	168	1060 STD/SSLT	1590 STD/SSLT
$^1/_4$	0.381	148	223		
$^5/_{16}$	0.476	184	277	975 OVS	1460 OVS
$^3/_8$	0.571	220	330		

	End-Plate	Beam
	F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

W40, 36, 33, 30, 27, 24, 21

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts
6 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$1/4$		$5/16$		$3/8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	93.5	140	117	175	140	210
	X	—	93.5	140	117	175	140	210
	SC Class A	STD	93.5	140	117	175	140	210
		OVS	83.7	126	105	157	117	175
		SSLT	93.5	140	117	175	137	206
	SC Class B	STD	93.5	140	117	175	140	210
		OVS	83.7	126	105	157	126	188
		SSLT	93.5	140	117	175	140	210
A490	N	—	93.5	140	117	175	140	210
	X	—	93.5	140	117	175	140	210
	SC Class A	STD	93.5	140	117	175	140	210
		OVS	83.7	126	105	157	126	188
		SSLT	93.5	140	117	175	140	210
	SC Class B	STD	93.5	140	117	175	140	210
		OVS	83.7	126	105	157	126	188
		SSLT	93.5	140	117	175	140	210

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
$3/16$	0.286	95.4	143
$1/4$	0.381	126	189
$5/16$	0.476	157	235
$3/8$	0.571	187	280

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
912	STD/ SSLT	1370	STD/ SSLT
839	OVS	1260	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

1-in.
Bolts
5 Rows

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

W30, 27,
24, 21,
18

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	77.2	116	96.5	145	116	174
	X	—	77.2	116	96.5	145	116	174
	SC Class A	STD	77.2	116	96.5	145	116	174
		OVS	69.1	104	86.3	129	97.2	146
		SSLT	77.2	116	96.5	145	114	171
	SC Class B	STD	77.2	116	96.5	145	116	174
		OVS	69.1	104	86.3	129	104	155
		SSLT	77.2	116	96.5	145	116	174
A490	N	—	77.2	116	96.5	145	116	174
	X	—	77.2	116	96.5	145	116	174
	SC Class A	STD	77.2	116	96.5	145	116	174
		OVS	69.1	104	86.3	129	104	155
		SSLT	77.2	116	96.5	145	116	174
	SC Class B	STD	77.2	116	96.5	145	116	174
		OVS	69.1	104	86.3	129	104	155
		SSLT	77.2	116	96.5	145	116	174

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	78.7	118
1/4	0.381	104	156
5/16	0.476	129	193
3/8	0.571	153	230

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
761	STD/ SSLT	1140	STD/ SSLT
702	OVS	1050	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load
N = Threads included
X = Threads excluded
SC = Slip critical

W24, 21, 18, 16

Table 10-4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts 4 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			1/4		5/16		3/8	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	60.9	91.4	76.1	114	91.4	137
	X	—	60.9	91.4	76.1	114	91.4	137
	SC Class A	STD	60.9	91.4	76.1	114	91.4	137
		OVS	54.4	81.6	68.0	102	77.7	117
		SSLT	60.9	91.4	76.1	114	91.4	137
	SC Class B	STD	60.9	91.4	76.1	114	91.4	137
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	60.9	91.4	76.1	114	91.4	137
A490	N	—	60.9	91.4	76.1	114	91.4	137
	X	—	60.9	91.4	76.1	114	91.4	137
	SC Class A	STD	60.9	91.4	76.1	114	91.4	137
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	60.9	91.4	76.1	114	91.4	137
	SC Class B	STD	60.9	91.4	76.1	114	91.4	137
		OVS	54.4	81.6	68.0	102	81.6	122
		SSLT	60.9	91.4	76.1	114	91.4	137

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
3/16	0.286	61.9	92.9
1/4	0.381	81.7	123
5/16	0.476	101	151
3/8	0.571	120	180

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
609	STD/ SSLT	914	STD/ SSLT
566	OVS	848	OVS

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical

**1-in.
Bolts
3 Rows**

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

**W18, 16,
14, 12,
10***

Bolt and End-Plate Available Strength, kips								
ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^1/_4$		$^5/_{16}$		$^3/_8$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	44.6	66.9	55.7	83.6	66.9	100
	X	—	44.6	66.9	55.7	83.6	66.9	100
	SC Class A	STD	44.6	66.9	55.7	83.6	66.9	100
		OVS	39.7	59.5	49.6	74.4	58.3	87.4
		SSLT	44.6	66.9	55.7	83.6	66.9	100
	SC Class B	STD	44.6	66.9	55.7	83.6	66.9	100
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	44.6	66.9	55.7	83.6	66.9	100
A490	N	—	44.6	66.9	55.7	83.6	66.9	100
	X	—	44.6	66.9	55.7	83.6	66.9	100
	SC Class A	STD	44.6	66.9	55.7	83.6	66.9	100
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	44.6	66.9	55.7	83.6	66.9	100
	SC Class B	STD	44.6	66.9	55.7	83.6	66.9	100
		OVS	39.7	59.5	49.6	74.4	59.5	89.3
		SSLT	44.6	66.9	55.7	83.6	66.9	100

Weld and Beam Web Available Strength, kips			
70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
$^3/_{16}$	0.286	45.2	67.9
$^1/_4$	0.381	59.4	89.1
$^5/_{16}$	0.476	73.1	110
$^3/_8$	0.571	86.3	129

Support Available Strength per Inch Thickness, kips/in.	
ASD	LRFD
458 STD/SSLT	687 STD/SSLT
429 OVS	644 OVS

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

N = Threads included
X = Threads excluded
SC = Slip critical
*Limited to W10×12, 15, 17, 19, 22, 26, 30

End-Plate	Beam
F_y = 36 ksi F_u = 58 ksi	F_y = 50 ksi F_u = 65 ksi

W12, 10, 8

Table 10–4 (continued)
Bolted/Welded Shear End-Plate Connections

1-in. Bolts 2 Rows

Bolt and End-Plate Available Strength, kips

ASTM Desig.	Thread Cond.	Hole Type	End-Plate Thickness, in.					
			$^{1}/_{4}$		$^{5}/_{16}$		$^{3}/_{8}$	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
A325/F1852	N	—	28.3	42.4	35.3	53.0	42.4	63.6
	X	—	28.3	42.4	35.3	53.0	42.4	63.6
	SC Class A	STD	28.3	42.4	35.3	53.0	42.4	63.6
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6
	SC Class B	STD	28.3	42.4	35.3	53.0	42.4	63.6
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6
A490	N	—	28.3	42.4	35.3	53.0	42.4	63.6
	X	—	28.3	42.4	35.3	53.0	42.4	63.6
	SC Class A	STD	28.3	42.4	35.3	53.0	42.4	63.6
		OVS	25.3	37.5	31.3	46.9	37.5	56.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6
	SC Class B	STD	28.3	42.4	35.3	53.0	42.4	63.6
		OVS	25.0	37.5	31.3	46.9	37.5	56.3
		SSLT	28.3	42.4	35.3	53.0	42.4	63.6

Weld and Beam Web Available Strength, kips

70 ksi Weld Size, in.	Minimum Beam Web Thickness, in.	R_n/Ω kips	ϕR_n kips
		ASD	LRFD
$^{3}/_{16}$	0.286	28.5	42.8
$^{1}/_{4}$	0.381	37.1	55.7
$^{5}/_{16}$	0.476	45.2	67.9
$^{3}/_{8}$	0.571	52.9	79.4

Support Available Strength per Inch Thickness, kips/in.

ASD		LRFD	
307	STD/ SSLT	461	STD/ SSLT
293	OVS	439	OVS

End-Plate	Beam
$F_y = 36$ ksi $F_u = 58$ ksi	$F_y = 50$ ksi $F_u = 65$ ksi

STD = Standard holes
OVS = Oversized holes
SSLT = Short-slotted holes transverse to direction of load

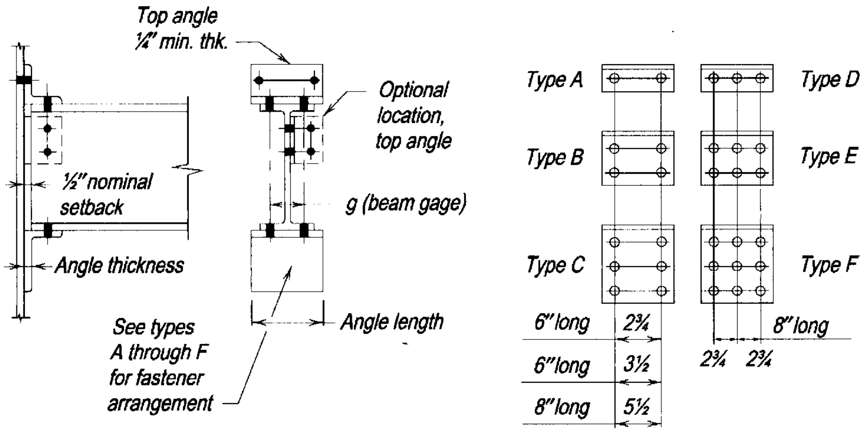
N = Threads included
X = Threads excluded
SC = Slip critical

UNSTIFFENED SEATED CONNECTIONS

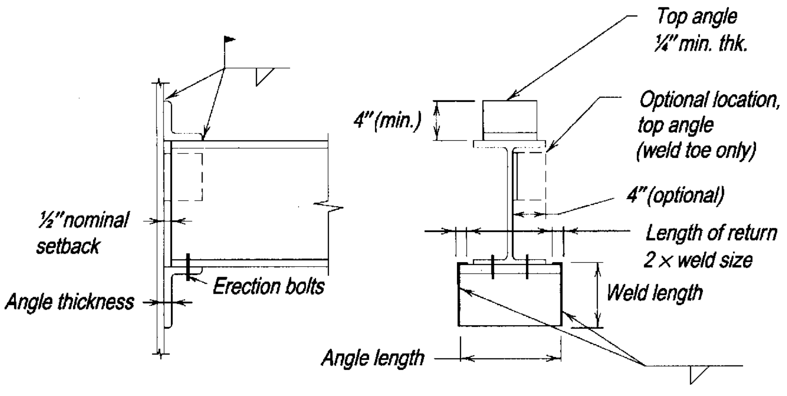
An unstiffened seated connection is made with a seat angle and a top angle, as illustrated in Figure 10–7. These angles may be bolted or welded to the supported beam as well as to the supporting member.

While the seat angle is assumed to carry the entire end reaction of the supported beam, the top angle must be placed as shown or in the optional side location for satisfactory performance and stability (Roeder and Dailey, 1989). The top angle and its connections are not usually sized for any calculated strength requirement. A $^1/_4$-in.-thick angle with a 4-in. vertical leg dimension will generally be adequate. It may be bolted with two bolts through each leg or welded with minimum-size welds to either the supported or the supporting members.

When the top angle is welded to the support and/or the supported beam, adequate flexibility must be provided in the connection. As illustrated in Figure 10–7b, line welds are placed along the toe of each angle leg. Note that welding along the sides of the vertical angle leg must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of such a connection would not be as intended for unstiffened seated connections.



(a) All-bolted



(b) All-welded

Figure 10–7. Unstiffened seated connections.

Design Checks

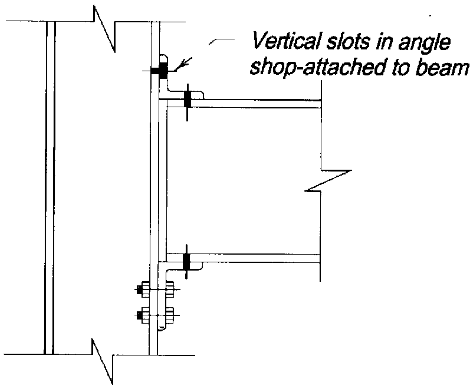
The available strength of an unstiffened seated connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, the strength of the supported beam web must be checked for the limit states of web local yielding and web local crippling. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

The available strength for web local yielding, ϕR_n or R_n/Ω, is determined per AISC Specification Section J10.2, which is simplified using the constants in Table 9-4. For further information, see Carter et al. (1997).

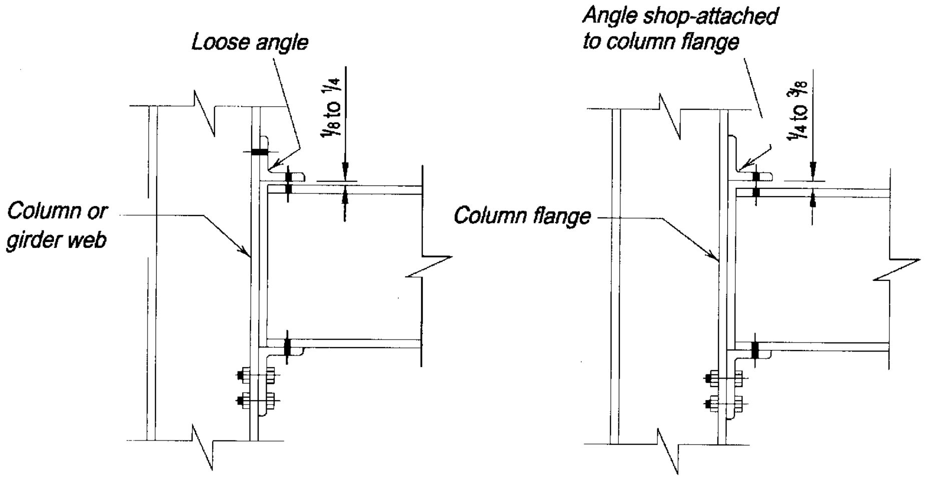
Shop and Field Practices

Unstiffened seated connections may be made to the webs and flanges of supporting columns. If adequate clearance exists, unstiffened seated connections may also be made to the webs of supporting girders.

To provide for overrun in beam length, the nominal setback for the beam end is 1/2 in. To provide for underrun in beam length, this setback is assumed to be 3/4 in. for calculation purposes.



(a) Vertical slots



(b) Loose angle with clearance as shown

(c) Shop-attached to column flange with clearance as shown

Figure 10–8. Providing for variation in beam depth with seated connections.

The seat angle is preferably shop-attached to the support. Since the bottom flange typically establishes the plane of reference for seated connections, mill variation in beam depth may result in variation in the elevation of the top flange. Such variation is usually of no consequence with concrete slab and metal deck floors, but may be a concern when a grating or steel-plate floor is used. Unless special care is required, the usual mill tolerances for member depth of $^1/_8$ in. to $^1/_4$ in. are ignored. However, when the top angle is shop-attached to the supported beam and field bolted to the support, mill variation in beam depth must be considered. Slotted holes, as illustrated in Figure 10–8a, will accommodate both overrun and underrun in the beam depth and are the preferred method for economy and convenience to both the fabricator and erector. Alternatively, the angle could be shipped loose with clearance provided, as shown in Figure 10–8b. When the top angle is to be field-welded to the support, no provision for mill variation in the beam depth is necessary.

When the top angle is shop-attached to the support, an appropriate erection clearance is provided, as illustrated in Figure 10–8c.

Table 10–5. All-Bolted Unstiffened Seated Connections

Table 10–5 is a design aid for all-bolted unstiffened seats. Seat available strengths are tabulated, assuming a 4-in. outstanding leg, for angle material with $F_y = 36$ ksi and $F_u = 58$ ksi and beam material with $F_y = 50$ ksi and $F_u = 65$ ksi. All values are for comparison with the LRFD load combination for LRFD design and the ASD load combination for ASD design.

Tabulated seat available strengths consider the limit states of shear yielding and flexural yielding of the outstanding angle leg, and local yielding and crippling of the beam web. A nominal beam setback of $^1/_2$ in. is assumed in these tables. However, this setback is increased to $^3/_4$ in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.

Bolt available strengths are tabulated for the seat types illustrated in Figure 10–8a with $^3/_4$-in., $^7/_8$-in., and 1-in. diameter ASTM A325, F1852 and A490 bolts. Vertical spacing of bolts and gages in seat angles may be arranged to suit conditions, provided the edge distance and spacing requirements in AISC Specification Section J3 are met. Where thick angles are used, larger entering and tightening clearances may be required in the outstanding angle leg. The suitability of angle sizes and thicknesses for the seat types illustrated in Figure 10–8a is also listed in Table 10–5.

Bolted/Welded Unstiffened Seated Connections

Tables 10–5 and 10–6 may be used in combination to design unstiffened seated connections that are welded to the supporting member and bolted to the supported beam, or bolted to the supporting member and welded to the supported beam.

Table 10–6. All-Welded Unstiffened Seated Connections

Table 10–6 is a design aid for all-welded unstiffened seats (exception: the beam is bolted to the seat). Seat available strengths are tabulated, assuming either a $3^1/_2$-in. or 4-in. outstanding leg (as indicated in the table), for angle material with $F_y = 36$ ksi and $F_u = 58$ ksi and beam material with $F_y = 50$ ksi and $F_u = 65$ ksi. Electrode strength is assumed to be 70 ksi.

Tabulated seat available strengths consider the limit states of shear yielding and flexural yielding of the outstanding angle leg, and local yielding and crippling of the beam web. A nominal beam setback of 1/2 in. is assumed in these tables. However, this setback is increased to 3/4 in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.

Weld available strengths are tabulated using the elastic method. The minimum and maximum angle thickness for each case is also tabulated. While these tabular values are based upon 70 ksi electrodes, they may be used for other electrodes, provided the tabular values are adjusted for the electrodes used (e.g., for 60 ksi electrodes, multiply the tabular values by 60/70 = 0.866, etc.) and the welds and base metal meet the required strength level provisions of AISC Specification Section J2. Should combinations of material thickness and weld size selected from Table 10–6 exceed the limits in AISC Specification Section J2.2, the weld size or material thickness should be increased as required.

As can be seen from the following, reduction of the tabulated weld strength is not normally required when unstiffened seats line up on opposite sides of the supporting web. From Salmon and Johnson (1996), the available strength, ϕR_n or R_n/Ω, of the welds to the support is

LRFD	ASD
$\phi R_n = 2\times\dfrac{1.392DL}{\sqrt{1+\dfrac{20.25e^2}{L^2}}}$	$\dfrac{R_n}{\Omega} = 2\times\dfrac{0.928DL}{\sqrt{1+\dfrac{20.25e^2}{L^2}}}$

where

D = number of sixteenths-of-an-inch in the weld size.

L = vertical leg dimension of the seat angle, in.

e = eccentricity of the beam end reaction with respect to the weld lines, in.

The term in the denominator that accounts for the eccentricity e increases the weld size far beyond what is required for shear alone, but with seats on both sides of the supporting member web, the forces due to eccentricity react against each other and have no effect on the web. Furthermore, as illustrated in Figure 10–9, there are actually two shear planes per weld, one at each weld toe and heel for a total of four shear planes. Thus, for an 8-in.-long 7×4×1 seat angle supporting a LRFD required strength of 70 kips or an equivalent ASD required strength of 46.67 kips, the minimum support thickness would be determined as follows

LRFD	ASD
$\dfrac{70\text{ kips}}{0.75\times 0.6\times 65\text{ ksi}\times 7\text{ in.}\times 4\text{ planes}} = 0.0855\text{ in.}$	$\dfrac{2.0\times 46.67\text{ kips}}{0.6\times 65\text{ ksi}\times 7\text{ in.}\times 4\text{ planes}} = 0.0855\text{ in.}$

For the identical connection on both sides of the support, the minimum support thickness would be less than 3/16 in. Thus, supporting web thickness is generally not a concern.

L6

Table 10-5
All-Bolted Unstiffened Seated Connections

Angle
F_y = 36 ksi

Outstanding Angle Leg Length Strength, kips

Angle Length, in.: 6

Required Bearing Length, N_{req}, in.	Angle Thickness, in.: 3/8		1/2		5/8		3/4		1		Min. Angle Leg
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	18.2	27.3									3 1/2
9/16	16.2	24.3									
5/8	14.6	21.9	43.1	64.8							
11/16	13.2	19.9	37.0	55.5							
3/4	12.1	18.2	32.3	48.6							
13/16	11.2	16.8	28.7	43.2							
7/8	10.4	15.6	25.9	38.9							
15/16	9.70	14.6	23.5	35.3	54.0	81.0					
1	9.09	13.7	21.6	32.4	50.5	75.9					
1 1/16	8.56	12.9	19.9	29.9	44.9	67.5					
1 1/8	8.08	12.2	18.5	27.8	40.4	60.8					
1 3/16	7.66	11.5	17.2	25.9	36.7	55.2					
1 1/4	7.28	10.9	16.2	24.3	33.7	50.6					
1 5/16	6.93	10.4	15.2	22.9	31.1	46.7	64.7	97.2			
1 3/8	6.61	9.94	14.4	21.6	28.9	43.4	58.2	87.5			
1 7/16	6.33	9.51	13.6	20.5	26.9	40.5	52.9	79.5			
1 1/2	6.06	9.11	12.9	19.4	25.3	38.0	48.5	72.9			
1 5/8	5.60	8.41	11.8	17.7	22.5	33.8	41.6	62.5			
1 3/4	5.20	7.81	10.8	16.2	20.2	30.4	36.4	54.7			
1 7/8	4.85	7.29	10.0	15.0	18.4	27.6	32.3	48.6			
2	4.55	6.83	9.24	13.9	16.8	25.3	29.1	43.7	86.2	130	
2 1/8	4.28	6.43	8.62	13.0	15.5	23.4	26.5	39.8	73.9	111	
2 1/4	4.04	6.08	8.08	12.2	14.4	21.7	24.3	36.5	64.7	97.2	
2 3/8	3.83	5.76	7.61	11.4	13.5	20.3	22.4	33.6	57.5	86.4	
2 1/2	3.64	5.47	7.19	10.8	12.6	19.0	20.8	31.2	51.7	77.8	
2 5/8	3.46	5.21	6.81	10.2	11.9	17.9	19.4	29.2	47.0	70.7	
2 3/4	3.31	4.97	6.47	9.72	11.2	16.9	18.2	27.3	43.1	64.8	
2 7/8	3.16	4.75	6.16	9.26	10.6	16.0	17.1	25.7	39.8	59.8	4
3	3.03	4.56	5.88	8.84	10.1	15.2	16.2	24.3	37.0	55.5	
3 1/8	2.91	4.37	5.62	8.45	9.62	14.5	15.3	23.0	34.5	51.8	
3 1/4	2.80	4.21	5.39	8.10	9.19	13.8	14.6	21.9	32.3	48.6	

Bolt Available Strength, kips

Bolt Dia., in.	ASTM Desig.	Thread Cond.	Connection Type from Figure 10-7a: A		B		C	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
3/4	A325/ F1852	N	21.2	31.8	42.4	63.6	63.6	95.4
		X	26.5	39.8	53.0	79.5	79.5	119
	A490	N	26.5	39.8	53.0	79.5	79.5	119
		X	33.1	49.7	66.3	99.4	99.4	149
7/8	A325/ F1852	N	28.9	43.3	57.7	86.6	86.6	130
		X	36.1	54.1	72.2	108	108	162
	A490	N	36.1	54.1	72.2	108	108	162
		X	45.1	67.6	90.2	135	135	203
1	A325/ F1852	N	37.7	56.5	75.4	113	—	—
		X	47.1	70.7	94.2	141	—	—
	A490	N	47.1	70.7	94.2	141	—	—
		X	58.9	88.4	118	177	—	—

Available Angles

Connection Type	Angle Size	t, in.
A, D	4×3	3/8 – 1/2
	4×3 1/2	3/8 – 1/2
	4×4	3/8 – 3/4
B, E	6×4	3/8 – 3/4
	7×4	3/8 – 3/4
	8×4	1/2 – 1
C, F[b]	8×4	1/2 – 1

[b] Not suitable for use with 1-in. diameter bolts.

ASD	LRFD
Ω = 2.00	ϕ = 0.75

For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

Angle
F_y = 36 ksi

Table 10–5 (continued)
All-Bolted Unstiffened Seated Connections

L8

Outstanding Angle Leg Length Strength, kips

Required Bearing Length N_{req}, in.	Angle Length, in. 8 — Angle Thickness, in. 3/8		1/2		5/8		3/4		1		Min. Angle Leg
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	24.3	36.5									3 1/2
9/16	21.6	32.4									
5/8	19.4	29.2	57.5	86.4							
11/16	17.6	26.5	49.3	74.1							
3/4	16.2	24.3	43.1	64.8							
13/16	14.9	22.4	38.3	57.6							
7/8	13.9	20.8	34.5	51.8							
15/16	12.9	19.4	31.4	47.1	72.0	108					
1	12.1	18.2	28.7	43.2	67.4	101					
1 1/16	11.4	17.2	26.5	39.9	59.9	90.0					
1 1/8	10.8	16.2	24.6	37.0	53.9	81.0					
1 3/16	10.2	15.3	23.0	34.6	49.0	73.6					
1 1/4	9.7	14.6	21.6	32.4	44.9	67.5					
1 5/16	9.2	13.9	20.3	30.5	41.5	62.3	86.2	130			
1 3/8	8.82	13.3	19.2	28.8	38.5	57.9	77.6	117			
1 7/16	8.44	12.7	18.2	27.3	35.9	54.0	70.5	106			
1 1/2	8.08	12.2	17.2	25.9	33.7	50.6	64.7	97.2			
1 5/8	7.46	11.2	15.7	23.6	29.9	45.0	55.4	83.3			
1 3/4	6.93	10.4	14.4	21.6	26.9	40.5	48.5	72.9			
1 7/8	6.47	9.72	13.3	19.9	24.5	36.8	43.1	64.8			
2	6.06	9.11	12.3	18.5	22.5	33.8	38.8	58.3	115	173	
2 1/8	5.71	8.58	11.5	17.3	20.7	31.2	35.3	53.0	98.5	148	
2 1/4	5.39	8.10	10.8	16.2	19.2	28.9	32.3	48.6	86.2	130	
2 3/8	5.11	7.67	10.1	15.2	18.0	27.0	29.8	44.9	76.6	115	
2 1/2	4.85	7.29	9.58	14.4	16.8	25.3	27.7	41.7	69.0	104	
2 5/8	4.62	6.94	9.08	13.6	15.9	23.8	25.9	38.9	62.7	94.3	
2 3/4	4.41	6.63	8.62	13.0	15.0	22.5	24.3	36.5	57.5	86.4	
2 7/8	4.22	6.34	8.21	12.3	14.2	21.3	22.8	34.3	53.1	79.8	4
3	4.04	6.08	7.84	11.8	13.5	20.3	21.6	32.4	49.3	74.1	
3 1/8	3.88	5.83	7.50	11.3	12.8	19.3	20.4	30.7	46.0	69.1	
3 1/4	3.73	5.61	7.19	10.8	12.2	18.4	19.4	29.2	43.1	64.8	

Bolt Available Strength, kips

Bolt Dia., in.	ASTM Desig.	Thread Cond.	Connection Type from Figure 10-7a: D		E		F	
			ASD	LRFD	ASD	LRFD	ASD	LRFD
3/4	A325/ F1852	N	31.8	47.7	63.6	95.4	95.4	143
		X	39.8	59.6	79.5	119	119	179
	A490	N	39.8	59.6	79.5	119	119	179
		X	49.7	74.6	99.4	149	149	224
7/8	A325/ F1852	N	43.3	64.9	86.6	130	130	195
		X	54.1	81.2	108	162	162	244
	A490	N	54.1	81.2	108	162	162	244
		X	67.6	101	135	203	203	304
1	A325/ F1852	N	56.5	84.8	113	170	—	—
		X	70.7	106	141	212	—	—
	A490	N	70.7	106	141	212	—	—
		X	88.4	133	177	265	—	—

Available Angles

Connection Type	Angle Size	t, in.
A, D	4×3	3/8 – 1/2
	4×3 1/2	3/8 – 1/2
	4×4	3/8 – 3/4
B, E	6×4	3/8 – 3/4
	7×4	3/8 – 3/4
	8×4	1/2 – 1
C, F[b]	8×4	1/2 – 1

[b] Not suitable for use with 1-in. diameter bolts.

ASD	LRFD
Ω = 2.00	ϕ = 0.75

For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

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Table 10-6
All-Welded Unstiffened Seated Connections

Angle
F_y = 36 ksi

Outstanding Angle Leg Length Strength, kips

Required Bearing Length N_{req}, in.	Angle Length, in.										Min. Angle Leg
	6										
	Angle Thickness, in.										
	3/8		1/2		5/8		3/4		1		
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	18.2	27.3									3 1/2
9/16	16.2	24.3									
5/8	14.6	21.9	43.1	64.8							
11/16	13.2	19.9	37.0	55.5							
3/4	12.1	18.2	32.3	48.6							
13/16	11.2	16.8	28.7	43.2							
7/8	10.4	15.6	25.9	38.9							
15/16	9.70	14.6	23.5	35.3	54.0	81.0					
1	9.09	13.7	21.6	32.4	50.5	75.9					
1 1/16	8.56	12.9	19.9	29.9	44.9	67.5					
1 1/8	8.08	12.2	18.5	27.8	40.4	60.8					
1 3/16	7.66	11.5	17.2	25.9	36.7	55.2					
1 1/4	7.28	10.9	16.2	24.3	33.7	50.6					
1 5/16	6.93	10.4	15.2	22.9	31.1	46.7	64.7	97.2			
1 3/8	6.61	9.94	14.4	21.6	28.9	43.4	58.2	87.5			
1 7/16	6.33	9.51	13.6	20.5	26.9	40.5	52.9	79.5			
1 1/2	6.06	9.11	12.9	19.4	25.3	38.0	48.5	72.9			
1 5/8	5.60	8.41	11.8	17.7	22.5	33.8	41.6	62.5			
1 3/4	5.20	7.81	10.8	16.2	20.2	30.4	36.4	54.7			
1 7/8	4.85	7.29	9.95	15.0	18.4	27.6	32.3	48.6			
2	4.55	6.83	9.24	13.9	16.8	25.3	29.1	43.7	86.2	130	
2 1/8	4.28	6.43	8.62	13.0	15.5	23.4	26.5	39.8	73.9	111	
2 1/4	4.04	6.08	8.08	12.2	14.4	21.7	24.3	36.5	64.7	97.2	
2 3/8	3.83	5.76	7.61	11.4	13.5	20.3	22.4	33.6	57.5	86.4	
2 1/2	3.64	5.47	7.19	10.8	12.6	19.0	20.8	31.2	51.7	77.8	
2 5/8	3.46	5.21	6.81	10.2	11.9	17.9	19.4	29.2	47.0	70.7	
2 3/4	3.31	4.97	6.47	9.72	11.2	16.9	18.2	27.3	43.1	64.8	
2 7/8	3.16	4.75	6.16	9.26	10.6	16.0	17.1	25.7	39.8	59.8	4
3	3.03	4.56	5.88	8.84	10.1	15.2	16.2	24.3	37.0	55.5	
3 1/8	2.91	4.37	5.62	8.45	9.62	14.5	15.3	23.0	34.5	51.8	
3 1/4	2.80	4.21	5.39	8.10	9.19	13.8	14.6	21.9	32.3	48.6	

Weld (70 ksi) Available Strength, kips

70 ksi Weld Size, in.	Seat Angle Size (long leg vertical)			
	4 × 3 1/2		5 × 3 1/2	
Design	ASD	LRFD	ASD	LRFD
1/4	11.5	17.2	17.2	25.8
5/16	14.3	21.5	21.5	32.2
3/8	17.2	25.8	25.8	38.7
7/16	20.1	30.1	30.1	45.2
1/2	—	—	34.4	51.6
9/16	—	—	38.7	58.1
5/8	—	—	43.0	64.5
11/16	—	—	47.3	71.0

Available Angle Thickness, in.

		4 × 3 1/2	5 × 3 1/2
Minimum		3/8	3/8
Maximum		1/2	3/4
ASD	LRFD	For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.	
Ω = 2.00	ϕ = 0.75		

Angle
F_y = 36 ksi

Table 10-6 (continued)
All-Welded Unstiffened Seated Connections

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Outstanding Angle Leg Length Strength, kips

Required Bearing Length N_{req}, in.	Angle Length, in.										Min. Angle Leg
	8										
	Angle Thickness, in.										
	3/8		1/2		5/8		3/4		1		
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	in.
1/2	24.3	36.5									3 1/2
9/16	21.6	32.4									
5/8	19.4	29.2	57.5	86.4							
11/16	17.6	26.5	49.3	74.1							
3/4	16.2	24.3	43.1	64.8							
13/16	14.9	22.4	38.3	57.6							
7/8	13.9	20.8	34.5	51.8							
15/16	12.9	19.4	31.4	47.1	72.0	108					
1	12.1	18.2	28.7	43.2	67.4	101					
1 1/16	11.4	17.2	26.5	39.9	59.9	90.0					
1 1/8	10.8	16.2	24.6	37.0	53.9	81.0					
1 3/16	10.2	15.3	23.0	34.6	49.0	73.6					
1 1/4	9.70	14.6	21.6	32.4	44.9	67.5					
1 5/16	9.24	13.9	20.3	30.5	41.5	62.3	86.2	130			
1 3/8	8.82	13.3	19.2	28.8	38.5	57.9	77.6	117			
1 7/16	8.44	12.7	18.2	27.3	35.9	54.0	70.5	106			
1 1/2	8.08	12.2	17.2	25.9	33.7	50.6	64.7	97.2			
1 5/8	7.46	11.2	15.7	23.6	29.9	45.0	55.4	83.3			
1 3/4	6.93	10.4	14.4	21.6	26.9	40.5	48.5	72.9			
1 7/8	6.47	9.72	13.3	19.9	24.5	36.8	43.1	64.8			
2	6.06	9.11	12.3	18.5	22.5	33.8	38.8	58.3	115	173	
2 1/8	5.71	8.58	11.5	17.3	20.7	31.2	35.3	53.0	98.5	148	
2 1/4	5.39	8.10	10.8	16.2	19.2	28.9	32.3	48.6	86.2	130	
2 3/8	5.11	7.67	10.1	15.2	18.0	27.0	29.8	44.9	76.6	115	
2 1/2	4.85	7.29	9.58	14.4	16.8	25.3	27.7	41.7	69.0	104	
2 5/8	4.62	6.94	9.08	13.6	15.9	23.8	25.9	38.9	62.7	94.3	
2 3/4	4.41	6.63	8.62	13.0	15.0	22.5	24.3	36.5	57.5	86.4	
2 7/8	4.22	6.34	8.21	12.3	14.2	21.3	22.8	34.3	53.1	79.8	4
3	4.04	6.08	7.84	11.8	13.5	20.3	21.6	32.4	49.3	74.1	
3 1/8	3.88	5.83	7.50	11.3	12.8	19.3	20.4	30.7	46.0	69.1	
3 1/4	3.73	5.61	7.19	10.8	12.2	18.4	19.4	29.2	43.1	64.8	

Weld (70 ksi) Available Strength, kips

70 ksi Weld Size, in.	Seat Angle Size (long leg vertical)					
	6 × 4		7 × 4		8 × 4	
Design	ASD	LRFD	ASD	LRFD	ASD	LRFD
1/4	21.8	32.7	28.5	42.7	35.6	53.4
5/16	27.3	40.9	35.6	53.4	44.5	66.7
3/8	32.7	49.1	42.7	64.1	53.4	80.1
7/16	38.2	57.2	49.8	74.7	62.3	93.4
1/2	43.6	65.4	57.0	85.4	71.2	107
9/16	49.1	73.6	64.1	96.1	80.1	120
5/8	54.5	81.8	71.2	107	89.0	133
11/16	60.0	90.0	78.3	117	97.9	147

Available Angle Thickness, in.

	6 × 4	7 × 4	8 × 4
Minimum	3/8	3/8	1/2
Maximum	3/4	3/4	1

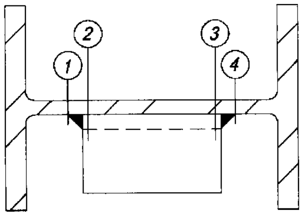
ASD	LRFD	
$\Omega = 2.00$	$\phi = 0.75$	For tabulated values above the heavy line, shear yielding of the angle leg controls the available strength.

STIFFENED SEATED CONNECTIONS

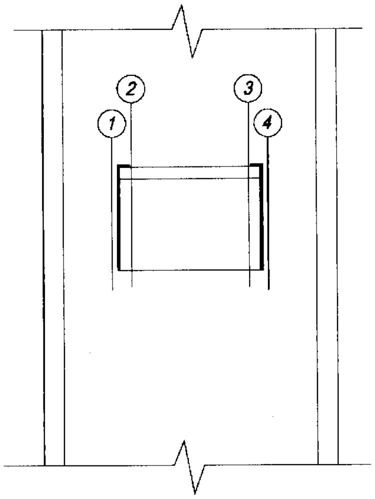
A stiffened seated connection is made with a seat plate and stiffening element (e.g., a plate, structural tee, or pair of angles) and a top angle, as illustrated in Figure 10–10. The top angle may be bolted or welded to the supported beam as well as to the supporting member and the stiffening element may be bolted or welded to the support. The seat plate should be bolted to the supported beam.

The stiffening element is assumed to carry the entire end reaction of the supported beam applied at a distance equal to $0.8W$. The top angle must be placed as shown or in the optional side location for satisfactory performance and stability (Roeder and Dailey, 1989). The top angle and its connections are not usually sized for any calculated strength requirement. A $^1/_4$-in.-thick angle with a 4-in. vertical leg dimension will generally be adequate. It may be bolted with two bolts through each leg or welded with minimum-size welds to either the supported or the supporting members.

When the top angle is welded to the support and/or the supported beam, adequate flexibility must be provided in the connection. As illustrated in Figure 10–10b, line welds are placed along the toe of each angle leg. Note that welding along the sides of the vertical angle leg must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of such a connection would not be as intended for simple shear connections.

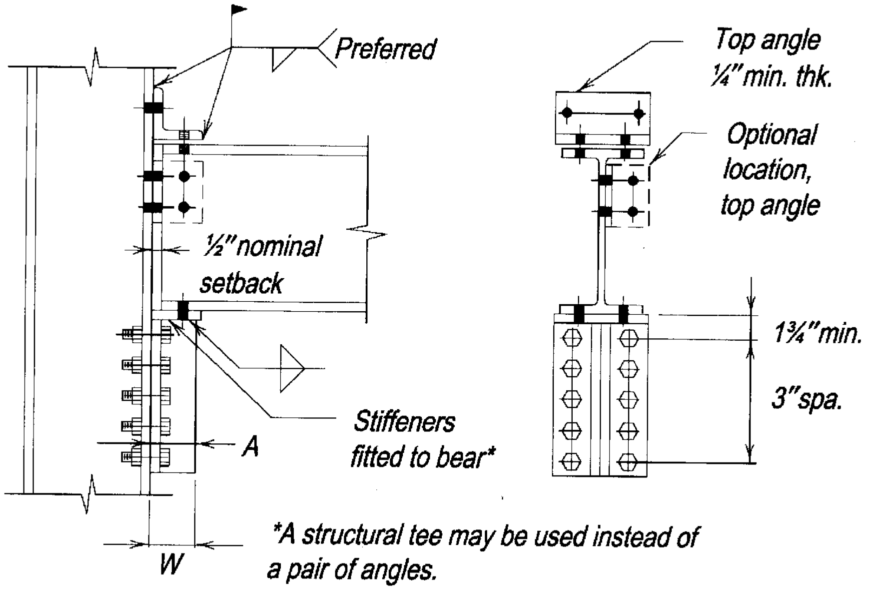


(a) Plan view



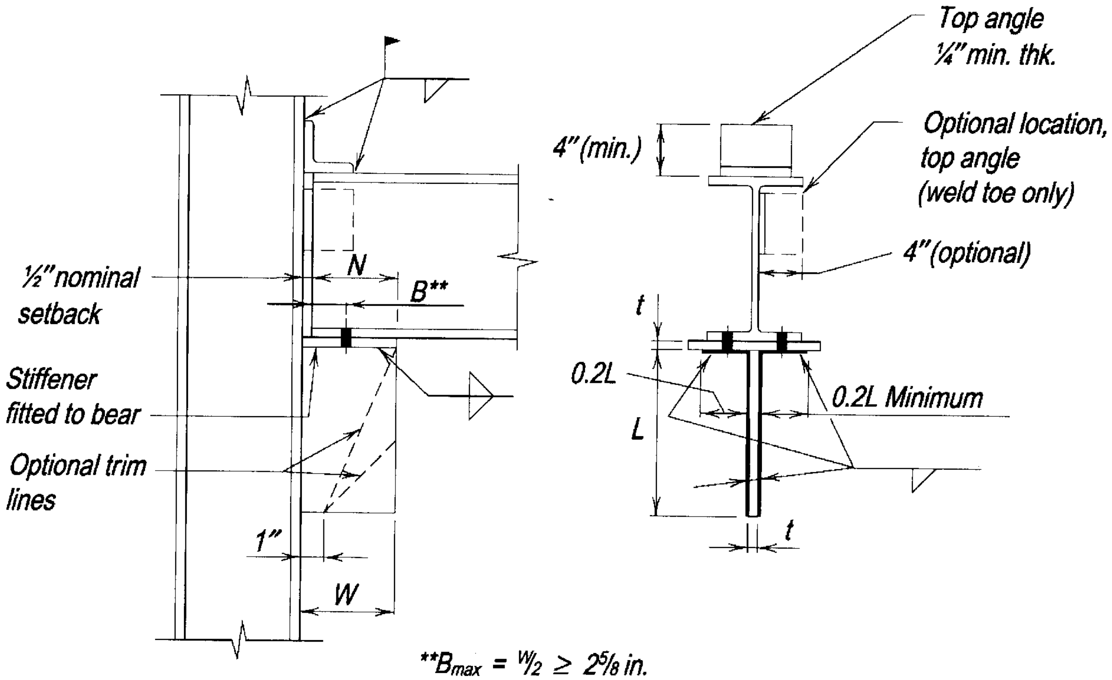
(b) Elevation

Figure 10–9. Shear planes in column web for unstiffened seated connections.



*A structural tee may be used instead of a pair of angles.

(a) All-bolted



**$B_{max} = W/2 \geq 2\tfrac{5}{8}$ in.

(b) Bolted/welded

Figure 10–10. Stiffened seated connections.

Design Checks

The available strength of a stiffened seated connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, the strength of the supported beam web must be checked for the limit states of local web yielding and web crippling. In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a. The available strength for web local yielding, ϕR_n or R_n/Ω, is determined per AISC Specification Section J10.2, which is simplified using the constants in Table 9–4.

When stiffened seated connections such as the one shown in Figure 10–10b are made to one side of a supporting column web, the column web may also need to be investigated for resistance to punching shear. In lieu of a more detailed analysis, Sputo and Ellifritt (1991) showed that punching shear will not be critical if the design parameters below and those summarized graphically in Figure 10–10b are met.

1. This simplified approach is applicable to the following column sections:

W14×43-808	W12×40-336	W10×33-112
W8×24-67	W6×20-25	W5×16-19

2. The supported beam must be bolted to the seat plate with high-strength bolts to account for the prying action caused by rotation of the connection. Welding the beam to the seat plate is not recommended because welds may lack the required strength and ductility. The centerline of the bolts should be located no more than the greater of $W/2$ or $2^5/_8$ in. from the column web face.
3. For seated connections where $W = 8$ in. or $W = 9$ in. and $3^1/_2$ in. $< B \leq W/2$, or where $W = 7$ in. and 3 in. $< B \leq W/2$ for a W14×43 column, refer to Sputo and Ellifritt (1991). These limitations are summarized at the bottom of Table 10–8.
4. The top angle may be bolted or welded, but must have a minimum $^1/_4$-in. thickness.
5. The seat plate should not be welded to the beam flange.

See also Ellifritt and Sputo (1999).

Shop and Field Practices

The comments for unstiffened seated connections are equally applicable to stiffened seated connections.

Table 10–7. All-Bolted Stiffened Seated Connections

Table 10–7 is a design aid for all-bolted stiffened seats. Stiffener available strengths are tabulated for stiffener material with $F_y = 36$ ksi and $F_u = 58$ ksi and with $F_y = 50$ ksi and $F_u = 65$ ksi.

Tabulated values consider the limit-state of bearing on the stiffening material. The designer must independently check the available strength of the beam web based upon the limit states of local web yielding and local web crippling. A nominal beam setback of $^1/_2$ in. is assumed in these tables. However, this setback is increased to $^3/_4$ in. for calculation purposes in determining the tabulated values to account for the possibility of underrun in beam length.

Bolt available strengths are tabulated for two vertical rows of from three to seven $^3/_4$-in., $^7/_8$-in., and 1-in.-diameter ASTM A325, F1852 and A490 high-strength bolts based upon the limit-state of bolt shear. Vertical spacing of bolts and gages in seat angles may be arranged to suit conditions, provided the edge distance and spacing requirements in AISC Specification Section J3 are met.

Table 10–8. Bolted/Welded Stiffened Seated Connections

Table 10–8 is a design aid for stiffened seated connections welded to the support and bolted to the supported beam. Electrode strength is assumed to be 70 ksi.

Weld available strengths are tabulated using the elastic method. While these tabular values are based upon 70 ksi electrodes, they may be used for other electrodes, provided the tabular values are adjusted for the electrodes used (e.g., for 60 ksi electrodes, multiply the tabular values by 60/70 = 0.866, etc.) and the weld and base metal meet the provisions of AISC Specification Section J2.

The thickness of the horizontal seat plate or tee flange should not be less than $^3/_8$ in. If the seat and stiffener are built up from separate plates, the stiffener should be finished to bear under the seat. The welds connecting the two plates should have a strength equal to or greater than the horizontal welds to the support under the seat plate.

The designer must independently check the beam web for web local yielding and web local crippling. The nominal beam setback of $^1/_2$ in. should be assumed to be $^3/_4$ in. for calculation purposes to account for possible underrun in beam length.

The stiffener thickness may be conservatively determined as follows. The minimum stiffener plate thickness, t, for supported beams with unstiffened webs should be the supported beam web thickness, t_w, multiplied by the ratio of F_y of the beam material to F_y of the stiffener material (e.g., F_y beam = 50 ksi, F_y stiffener = 36 ksi, $t = t_w \times 50/36$ minimum). Additionally, the minimum stiffener thickness, t, should be at least $2w$ for stiffener material with $F_y = 36$ ksi or $1.5w$ for stiffener material with $F_y = 50$ ksi, where w is the weld size for 70 ksi electrodes.

For 70 ksi electrodes, the minimum column web thickness is

$$t_{min} = \frac{3.09D}{F_u}$$

where

D = the weld size in sixteenths of an inch.

When welds line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the weld available strength must be reduced by the ratio of the thickness provided to the minimum thickness. As with unstiffened seated connections, the contribution of eccentricity to the required shear yielding strength is negligible. Should combinations of material thickness and weld size selected from Table 10–8 exceed the limits of AISC Specification Section J2, increase the weld size or material thickness as required.

Table 10–7

All-Bolted Stiffened Seated Connections

Stiffener Material		Outstanding Angle Leg Available Strength, kips[a]											
Stiffener Outstanding Leg A, in.[b]		$F_y = 36$ ksi						$F_y = 50$ ksi					
		3 1/2		4		5		3 1/2		4		5	
		ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
Thickness of Stiffener Outstanding Legs, in.	5/16	55.7	83.5	65.8	98.7	86.1	129	77.3	116	91.4	137	120	179
	3/8	66.8	100	79.0	118	103	155	92.8	139	110	165	143	215
	1/2	89.1	134	105	158	138	207	124	186	146	219	191	287
	5/8	111	167	132	197	172	258	155	232	183	274	239	359
	3/4	134	200	158	237	207	310	186	278	219	329	287	430

Use minimum 3/8-in. thick seat plate wide enough to extend beyond outstanding legs of stiffener.

[a] See AISC Specification Sect. J7.

[b] Beam bearing length assumed 3/4 in. less for calculation purposes.

Bolt Available Strength, kips												
Bolt Diameter, in.	ATSM Desig.	Thread Cond.	Number of Bolts in One Vertical Row									
			3		4		5		6		7	
			ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
3/4	A325/ F182	N	63.6	95.4	84.8	127	106	159	127	191	148	223
		X	79.5	119	106	159	133	199	159	239	186	278
	A490	N	79.5	119	106	159	133	199	159	239	186	278
		X	99.4	149	133	199	166	249	199	298	232	348
7/8	A325/ F182	N	86.6	130	115	173	144	216	173	260	202	303
		X	108	162	144	216	180	271	216	325	253	379
	A490	N	108	162	144	216	180	271	216	325	253	379
		X	135	203	180	271	225	338	271	406	316	474
1	A325/ F182	N	113	170	151	226	188	283	226	339	264	396
		X	141	212	188	283	236	353	283	424	330	495
	A490	N	141	212	188	283	236	353	283	424	330	495
		X	177	265	236	353	295	442	353	530	412	619

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$
$\dfrac{R_n}{\Omega} = \dfrac{1.8F_yA_{pb}}{2.00}$	$\phi R_n = 0.75 \times 1.8F_yA_{pb}$

Table 10–8
Bolted/Welded Stiffened Seated Connections

L, in.	Width of Seat *W*, in.											
	4								5			
	70 ksi Weld Size, in.								70 ksi Weld Size, in.			
	1/4		5/16		3/8		7/16		5/16		3/8	
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
6	**22.7**	34.0	**28.4**	42.5	**34.0**	51.1	**39.7**	59.6	**23.5**	35.2	**28.2**	42.2
7	**29.9**	44.9	**37.4**	56.1	**44.9**	67.3	**52.4**	78.6	**31.2**	46.9	**37.5**	56.2
8	**37.8**	56.7	**47.2**	70.8	**56.7**	85.0	**66.1**	99.2	**39.8**	59.8	**47.8**	71.7
9	**46.1**	69.2	**57.7**	86.5	**69.2**	104	**80.7**	121	**49.1**	73.7	**59.0**	88.5
10	**54.9**	82.3	**68.6**	103	**82.3**	123	**96.0**	144	**59.0**	88.5	**70.8**	106
11	**63.9**	95.8	**79.8**	120	**95.8**	144	**112**	168	**69.4**	104	**83.3**	125
12	**73.1**	110	**91.4**	137	**110**	165	**128**	192	**80.2**	120	**96.2**	144
13	**82.5**	124	**103**	155	**124**	186	**144**	217	**91.3**	137	**110**	164
14	**92.1**	138	**115**	173	**138**	207	**161**	242	**103**	154	**123**	185
15	**102**	152	**127**	191	**152**	229	**178**	267	**114**	171	**137**	206
16	**111**	167	**139**	209	**167**	250	**195**	292	**126**	189	**151**	227
17	**121**	181	**151**	227	**181**	272	**212**	318	**138**	207	**165**	248
18	**131**	196	**163**	245	**196**	294	**229**	343	**150**	225	**180**	270
19	**140**	211	**175**	263	**211**	316	**246**	369	**162**	243	**194**	291
20	**150**	225	**188**	281	**225**	338	**263**	394	**174**	261	**209**	313
21	**160**	240	**200**	300	**240**	359	**280**	419	**186**	279	**223**	335
22	**169**	254	**212**	318	**254**	381	**296**	445	**198**	297	**238**	357
23	**179**	269	**224**	336	**269**	403	**313**	470	**210**	315	**252**	378
24	**189**	283	**236**	354	**283**	425	**330**	495	**222**	334	**267**	400
25	**198**	297	**248**	372	**297**	446	**347**	520	**235**	352	**281**	422
26	**208**	312	**260**	390	**312**	468	**364**	546	**247**	370	**296**	444
27	**217**	326	**272**	408	**326**	489	**380**	571	**259**	388	**310**	466

Limitations for Connections to Column Webs	
B = 2⁵/₈ in. max	**B = 2⁵/₈ in. max**
W12×40, W14×43 for L ≥ 9 in. limit weld ≤ ¹/₄ in.	None

Notes:

1. Values shown assume 70 ksi electrodes. For 60 ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80 ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \frac{F_{y\ beam}}{F_{y\ stiffener}} \times t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.

3. Tabulated values may be limited by shear yielding of or bearing on the stiffener; refer to LRFD Specification Sections F2.2 and J8, respectively.

ASD	**LRFD**
$\Omega = 2.00$	$\phi = 0.75$

Table 10–8 (continued)

Bolted/Welded Stiffened Seated Connections

L, in.	**Width of Seat W, in.**											
	5				**6**							
	70 ksi Weld Size, in.				**70 ksi Weld Size, in.**							
	$^7/_{16}$		$^1/_2$		$^5/_{16}$		$^3/_8$		$^7/_{16}$		$^1/_2$	
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
6	**32.8**	49.3	**37.5**	56.3	**19.9**	29.9	**23.9**	35.9	**27.9**	41.9	**31.9**	47.8
7	**43.7**	65.6	**50.0**	75.0	**26.7**	40.1	**32.0**	48.1	**37.4**	56.1	**42.7**	64.1
8	**55.8**	83.7	**63.8**	95.6	**34.3**	51.4	**41.1**	61.7	**48.0**	72.0	**54.8**	82.2
9	**68.8**	103	**78.6**	118	**42.5**	63.8	**51.1**	76.6	**59.6**	89.3	**68.1**	102
10	**82.6**	124	**94.4**	142	**51.4**	77.2	**61.7**	92.6	**72.0**	108	**82.3**	123
11	**97.2**	146	**111**	167	**60.9**	91.3	**73.1**	110	**85.3**	128	**97.4**	146
12	**112**	168	**128**	192	**70.8**	106	**85.0**	127	**99.2**	149	**113**	170
13	**128**	192	**146**	219	**81.2**	122	**97.4**	146	**114**	170	**130**	195
14	**144**	216	**164**	246	**91.9**	138	**110**	165	**129**	193	**147**	220
15	**160**	240	**183**	274	**103**	154	**123**	185	**144**	216	**165**	247
16	**176**	265	**202**	302	**114**	171	**137**	205	**160**	240	**183**	274
17	**193**	290	**221**	331	**126**	188	**151**	226	**176**	264	**201**	301
18	**210**	315	**240**	360	**137**	206	**165**	247	**192**	288	**219**	329
19	**227**	340	**259**	388	**149**	223	**179**	268	**208**	313	**238**	357
20	**244**	365	**278**	417	**161**	241	**193**	289	**225**	337	**257**	386
21	**260**	391	**298**	446	**173**	259	**207**	311	**242**	362	**276**	414
22	**277**	416	**317**	476	**185**	277	**222**	332	**258**	388	**295**	443
23	**294**	442	**336**	505	**197**	295	**236**	354	**275**	413	**315**	472
24	**311**	467	**356**	534	**209**	313	**250**	376	**292**	438	**334**	501
25	**328**	492	**375**	563	**221**	331	**265**	397	**309**	464	**353**	530
26	**345**	518	**395**	592	**233**	349	**280**	419	**326**	489	**373**	559
27	**362**	543	**414**	621	**245**	368	**294**	441	**343**	515	**392**	588

Limitations for Connections to Column Webs	
B = 2$^5/_8$ in. max	**B = 3 in. max**
None	None

Notes:

1. Values shown assume 70 ksi electrodes. For 60 ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80 ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \frac{F_{y\ beam}}{F_{y\ stiffener}} \times t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.

3. Tabulated values may be limited by shear yielding of or bearing on the stiffener; refer to LRFD Specification Sections F2.2 and J8, respectively.

ASD	**LRFD**
$\Omega = 2.00$	$\phi = 0.75$

Table 10–8 (continued)
Bolted/Welded Stiffened Seated Connections

L, in.	Width of Seat *W*, in.											
	7								8			
	70 ksi Weld Size, in.								70 ksi Weld Size, in.			
	5/16		3/8		7/16		1/2		5/16		3/8	
	ASD	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
11	**54.0**	81.0	**64.8**	97.2	**75.6**	113	**86.4**	130	**48.4**	72.5	**58.0**	87.1
12	**63.1**	94.7	**75.7**	114	**88.4**	133	**101**	151	**56.7**	85.1	**68.1**	102
13	**72.7**	109	**87.2**	131	**102**	153	**116**	174	**65.6**	98.3	**78.7**	118
14	**82.6**	124	**99.2**	149	**116**	174	**132**	198	**74.8**	112	**89.8**	135
15	**93.0**	139	**112**	167	**130**	195	**149**	223	**84.5**	127	**101**	152
16	**104**	155	**124**	186	**145**	217	**166**	249	**94.4**	142	**113**	170
17	**114**	172	**137**	206	**160**	240	**183**	275	**105**	157	**126**	189
18	**126**	188	**151**	226	**176**	264	**201**	301	**115**	173	**138**	208
19	**137**	205	**164**	246	**192**	287	**219**	329	**126**	189	**151**	227
20	**148**	223	**178**	267	**208**	312	**237**	356	**137**	206	**165**	247
21	**160**	240	**192**	288	**224**	336	**256**	384	**148**	222	**178**	267
22	**172**	258	**206**	309	**240**	361	**275**	412	**160**	240	**192**	287
23	**184**	275	**220**	330	**257**	385	**294**	440	**171**	257	**205**	308
24	**195**	293	**234**	352	**274**	410	**313**	469	**183**	274	**219**	329
25	**207**	311	**249**	373	**290**	435	**332**	498	**195**	292	**233**	350
26	**219**	329	**263**	395	**307**	461	**351**	526	**206**	309	**248**	371
27	**231**	347	**278**	417	**324**	486	**370**	555	**218**	327	**262**	393
28	**244**	365	**292**	438	**341**	511	**390**	584	**230**	345	**276**	414
29	**256**	383	**307**	460	**358**	537	**409**	613	**242**	363	**291**	436
30	**268**	402	**321**	482	**375**	562	**428**	643	**254**	381	**305**	457
31	**280**	420	**336**	504	**392**	588	**448**	672	**266**	399	**319**	479
32	**292**	438	**350**	526	**409**	613	**467**	701	**278**	417	**334**	501

Limitations for Connections to Column Webs	
B = 3½ in. max	**B = 3½ in. max**
W14 × 43, limit B ≤ 3 in. See item 3 in preceding discussion "Design Checks"	See item 3 in preceding discussion "Design Checks"

Notes:

1. Values shown assume 70 ksi electrodes. For 60 ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80 ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \frac{F_{y\,beam}}{F_{y\,stiffener}} \times t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.

3. Tabulated values may be limited by shear yielding of or bearing on the stiffener; refer to LRFD Specification Sections F2.2 and J8, respectively.

ASD	**LRFD**
$\Omega = 2.00$	$\phi = 0.75$

Table 10-8 (continued)
Bolted/Welded Stiffened Seated Connections

L, in.	Width of Seat *W*, in.											
	8				9							
	70 ksi Weld Size, in.				70 ksi Weld Size, in.							
	$^1/_2$		$^5/_8$		$^5/_{16}$		$^3/_8$		$^1/_2$		$^5/_8$	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
11	**77.4**	116	**96.7**	145	**43.7**	65.6	**52.5**	78.7	**69.9**	105	**87.4**	131
12	**90.8**	136	**113**	170	**51.4**	77.1	**61.7**	92.5	**82.2**	123	**103**	154
13	**105**	157	**131**	197	**59.6**	89.3	**71.5**	107	**95.3**	143	**119**	179
14	**120**	180	**150**	224	**68.2**	102	**81.8**	123	**109**	164	**136**	204
15	**135**	203	**169**	253	**77.2**	116	**92.6**	139	**123**	185	**154**	232
16	**151**	227	**189**	283	**86.5**	130	**104**	156	**138**	208	**173**	260
17	**168**	251	**209**	314	**96.2**	144	**115**	173	**154**	231	**192**	289
18	**184**	277	**231**	346	**106**	159	**127**	191	**170**	255	**212**	319
19	**202**	303	**252**	378	**117**	175	**140**	210	**186**	280	**233**	350
20	**219**	329	**274**	411	**127**	191	**152**	229	**203**	305	**254**	381
21	**237**	356	**297**	445	**138**	207	**165**	248	**220**	331	**276**	413
22	**256**	383	**319**	479	**149**	223	**178**	268	**238**	357	**297**	446
23	**274**	411	**342**	514	**160**	240	**192**	288	**256**	384	**320**	480
24	**292**	439	**366**	548	**171**	257	**205**	308	**274**	411	**342**	513
25	**311**	467	**389**	584	**183**	274	**219**	329	**292**	438	**365**	548
26	**330**	495	**413**	619	**194**	291	**233**	349	**310**	466	**388**	582
27	**349**	524	**436**	655	**206**	308	**247**	370	**329**	494	**411**	617
28	**368**	552	**460**	690	**217**	326	**261**	391	**348**	522	**435**	652
29	**387**	581	**484**	726	**229**	344	**275**	412	**367**	550	**458**	687
30	**407**	610	**508**	762	**241**	362	**289**	434	**386**	578	**482**	723
31	**426**	639	**532**	799	**253**	379	**304**	455	**405**	607	**506**	759
32	**445**	668	**557**	835	**265**	397	**318**	477	**424**	636	**530**	795

Limitations for Connections to Column Webs	
B = $3^1/_2$ in. max	B = $3^1/_2$ in. max
See item 3 in preceding discussion "Design Checks"	See item 3 in preceeding discussion "Design Checks"

Notes:

1. Values shown assume 70 ksi electrodes. For 60 ksi electrodes, multiply tabular values by 0.857, or enter table with 1.17 times the required strength, R_u or R_a. For 80 ksi electrodes, multiply tabular values by 1.14, or enter table with 0.875 times the required strength.
2. Tabulated values are valid for stiffeners with minimum thickness of

$$t_{min} = \frac{F_{y\ beam}}{F_{y\ stiffener}} \times t_w$$

but not less than $2w$ for stiffeners with $F_y = 36$ ksi nor $1.5w$ for stiffeners with $F_y = 50$ ksi. In the above, t_w is the thickness of the unstiffened supported beam web and w is the nominal weld size.

3. Tabulated values may be limited by shear yielding of or bearing on the stiffener; refer to LRFD Specification Sections F2.2 and J8, respectively.

ASD	LRFD
$\Omega = 2.00$	$\phi = 0.75$

SINGLE-PLATE CONNECTIONS

A single-plate connection is made with a plate, as illustrated in Figure 10–11. The plate is always welded to the support on both sides of the plate and bolted to the supported member.

Design Checks

The available strength of a single-plate connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a, respectively.

Single-plate shear connections that satisfy the corresponding dimensional limitations can be designed using the simplified design procedure for the "conventional" configuration. Other single-plate shear connections can be designed using the procedure for the "extended" configuration, which is applicable to any configuration of single-plate shear connections, regardless of connection geometry.

Both the conventional and extended configurations permit the use of ASTM A325, F1852, or A490 bolts. The procedure is valid for bolts that are snug-tightened, pretensioned, or slip-critical. In both the conventional and extended configuration, the design recommendations are equally applicable to plate and beam web material with F_y = 36 ksi or 50 ksi. In both cases, the weld between the single plate and the support should be sized as $^{5}/_{8}t_p$, which will develop the strength of either a 36 ksi or 50 ksi plate.

Conventional Configuration

The following method may be used when the dimensional and other limitations upon which it is based are satisfied.

Dimensional Limitations

1. Only a single vertical row of bolts is permitted. The number of bolts in the connection, n, is limited to 2 to 12.
2. The distance from the bolt line to the weld line, a, must be equal to or less than $3^{1}/_{2}$ in.

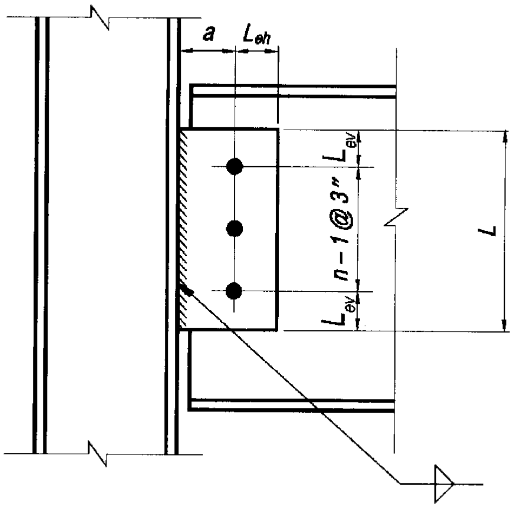


Figure 10–11. Single-plate connection.

3. STD or SSL holes are permitted to be used.
4. The horizontal edge distance, L_{eh}, must be equal to or greater than $2d_b$ for both the plate and the beam web. Note that L_{eh} is measured to the center of the hole or slot.
5. The vertical edge distance, L_{ev}, must satisfy AISC Specification Table J3.4 requirements.
6. Either the plate or the beam web must satisfy $t \leq d_b/2 + {}^1\!/_{16}$ in.

Design Checks

1. The connection must be checked for bolt shear, block shear rupture, and bolt bearing. For STD holes, eccentricity can be ignored when the number of bolts, n, is less than or equal to 9. For connections with 10 to 12 bolts, use $e = n - 4$ and a 1.25 multiplier on the calculated eccentricity coefficient C. For SSL holes, eccentricity can be ignored up to $n = 12$.
2. Check the plate for shear yielding and shear rupture. Plate buckling will not control for the conventional configuration.

Extended Configuration

The following method is useful when the dimensional and other limitations of the conventional method cannot be satisfied. This procedure can be used to determine the strength of single plate shear connections with multiple vertical rows or in the extended configuration, as shown in Figure 10–12.

Dimensional Limitations

1. The number of bolts, n, is not limited.
2. The distance from the bolt line to the weld line, a, is not limited.
3. The use of holes must satisfy AISC Specification Section J3.2 requirements.
4. The horizontal and vertical edge distances, L_{eh} and L_{ev}, must satisfy AISC Specification Table J3.4 requirements.

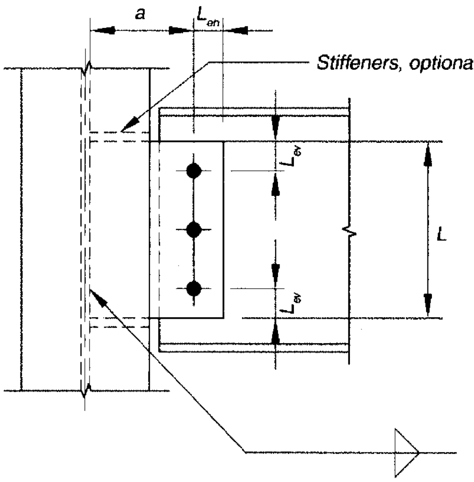


Figure 10–12. Extended single-plate connection.

Design Checks

1. Determine the bolt group required for bolt shear and bolt bearing with eccentricity $e = a$, where a is defined as the distance from the support to the first row of bolts. Exception: alternative considerations of the design eccentricity are acceptable when justified by rational analysis. For example, see Sherman and Ghorbanpoor (2002).
2. Determine the maximum plate thickness permitted such that the plate moment strength does not exceed the moment strength of the bolt group in shear, as follows:

$$t_{max} = \frac{6M_{max}}{F_y d^2}$$

where

$M_{max} = 1.25F_vA_bC'$

$1.25F_v$ = shear strength of an individual bolt from AISC Specification Table J3.2, ksi, multiplied by a factor of 1.25 to remove the 20 percent reduction for uneven for distribution in end-loaded bolt groups (Kulak, 2002). The joint in question is not end-loaded.

A_b = area of an individual bolt, in.^{2}

C' = coefficient from Part 7 for the moment-only case (instantaneous center of rotation at the centroid of the bolt group)

F_y = plate specified yield stress, ksi

d = plate depth, in.

The foregoing check is made at the nominal strength level, since the check is to ensure ductility, not strength.

Exceptions:

a. For a single vertical row of bolts only, the foregoing criterion need not be satisfied if either the beam web or the plate satisfies $t \leq d_b/2 + {}^1/_{16}$ and both satisfy $L_{eh} \geq 2d_b$.

b. For a double vertical row of bolts only, the foregoing criterion need not be satisfied if both the beam web and the plate satisfy $t \leq d_b/2 + {}^1/_{16}$ and $L_{eh} \geq 2d_b$.

3. Check the plate for shear yielding, shear rupture, and block shear rupture.
4. Check the plate for flexure with the von-Mises shear reduction. That is, check the available flexural yielding strength of the plate, ϕM_n or M_n/Ω, based upon a critical stress, F_{cr}, as follows:

$$F_{cr} = \sqrt{F_y^2 - 3f_v^2}$$

$$M_n = F_{cr}Z$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

5. Check the plate for buckling using the double-coped beam procedure given in Part 9.
6. Ensure that the supported beam is braced at points of support.

The design procedure for extended single-plate shear connections permits the column to be designed for an axial force without eccentricity. In some cases, economy may be gained by considering alternative design procedures that allow the transfer of some moment into the support, for example, 5 percent of the beam fixed-end moment, provided that this moment is also considered in the design of the supporting member.

Recommended Plate Length

To provide for stability during erection, it is recommended that the minimum plate length be one-half the T-dimension of the beam to be supported. The maximum length of the plate must be compatible with the T-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam. Note that the plate may encroach upon the fillet(s) as given in Figure 10–3.

Shop and Field Practices

Conventional and extended single-plate connections may be made to the webs of supporting girders and to the flanges of supporting columns. Extended single-plate connections are suitable for connections to the webs of supporting columns when the bolt line is located a sufficient distance beyond the column flanges.

With the plate shop-attached to the support, side erection of the beam is permitted. Play in the open holes usually compensates for mill variation in column flange supports and other field adjustments.

Table 10–9. Single-Plate Connections

Table 10–9 is a design aid for single-plate connections welded to the support and bolted to the supported beam. Available strengths are tabulated for plate material with $F_y = 36$ ksi and $F_u = 58$ ksi.

Tabulated bolt and plate available strengths consider the limit-states of bolt shear, bolt bearing on the plate, shear yielding of the plate, shear rupture of the plate, block shear rupture of the plate, and weld shear. Values are tabulated for two through twelve rows of $^3/_4$-in., $^7/_8$-in., 1-in. and $1^1/_8$-in. diameter ASTM A325, F1852, and A490 bolts at 3-in. spacing. For calculation purposes, plate edge distance, L_{ev}, is in accordance with AISC Specification Section J3.10 and Table J3.4. End distance, L_{eh}, is provided as 2 times the diameter of the bolt, to match tested connections. Weld sizes are tabulated equal to $^5/_8 t_p$.

While the tabular values are based on $a = 3$ in., they may conservatively be used when the distance from the support to the bolt line, a, is between $2^1/_2$ in. and 3 in. The tabulated values are valid for laterally supported beams in steel and composite construction, all types of loading, snug-tightened or pretensioned bolts, and for supported and supporting members of all grades of steel.

Plate
F_y = 36 ksi

Table 10-9a
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

3/4-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (*L* = 35 1/2)	A325 F1852	N	STD	100	150	118	178	118	178	118	178	—	—	—	—
			SSLT	99.5	149	124	187	127	191	127	191	—	—	—	—
		X	STD	100	150	125	188	148	222	148	222	—	—	—	—
			SSLT	99.5	149	124	187	149	224	159	239	—	—	—	—
	A490	N	STD	100	150	125	188	148	222	148	222	—	—	—	—
			SSLT	99.5	149	124	187	149	224	159	239	—	—	—	—
		X	STD	100	150	125	188	150	225	175	263	—	—	—	—
			SSLT	99.5	149	124	187	149	224	174	261	—	—	—	—
11 (*L* = 32 1/2)	A325 F1852	N	STD	92.1	138	111	166	111	166	111	166	—	—	—	—
			SSLT	91.4	137	114	171	117	175	117	175	—	—	—	—
		X	STD	92.1	138	115	173	138	207	139	208	—	—	—	—
			SSLT	91.4	137	114	171	137	206	146	219	—	—	—	—
	A490	N	STD	92.1	138	115	173	138	207	139	208	—	—	—	—
			SSLT	91.4	137	114	171	137	206	146	219	—	—	—	—
		X	STD	92.1	138	115	173	138	207	161	242	—	—	—	—
			SSLT	91.4	137	114	171	137	206	160	240	—	—	—	—
10 (*L* = 29 1/2)	A325 F1852	N	STD	84.0	126	103	155	103	155	103	155	—	—	—	—
			SSLT	83.3	125	104	156	106	159	106	159	—	—	—	—
		X	STD	84.0	126	105	157	126	189	129	194	—	—	—	—
			SSLT	83.3	125	104	156	125	187	133	199	—	—	—	—
	A490	N	STD	84.0	126	105	157	126	189	129	194	—	—	—	—
			SSLT	83.3	125	104	156	125	187	133	199	—	—	—	—
		X	STD	84.0	126	105	157	126	189	147	220	—	—	—	—
			SSLT	83.3	125	104	156	125	187	146	219	—	—	—	—
9 (*L* = 26 1/2)	A325 F1852	N	STD	75.9	114	94.8	142	95.4	143	95.4	143	—	—	—	—
			SSLT	75.2	113	94.0	141	95.4	143	95.4	143	—	—	—	—
		X	STD	75.9	114	94.8	142	114	171	119	179	—	—	—	—
			SSLT	75.2	113	94.0	141	113	169	119	179	—	—	—	—
	A490	N	STD	75.9	114	94.8	142	114	171	119	179	—	—	—	—
			SSLT	75.2	113	94.0	141	113	169	119	179	—	—	—	—
		X	STD	75.9	114	94.8	142	114	171	133	199	—	—	—	—
			SSLT	75.2	113	94.0	141	113	169	132	197	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

3/4-in. diameter bolts

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 36 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
8 ($L = 23\frac{1}{2}$)	A325 F1852	N	STD	67.8	102	84.7	127	84.8	127	84.8	127	—	—	—	—
			SSLT	67.1	101	83.9	126	84.8	127	84.8	127	—	—	—	—
		X	STD	67.8	102	84.7	127	102	153	106	159	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	106	159	—	—	—	—
	A490	N	STD	67.8	102	84.7	127	102	153	106	159	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	106	159	—	—	—	—
		X	STD	67.8	102	84.7	127	102	153	119	178	—	—	—	—
			SSLT	67.1	101	83.9	126	101	151	117	176	—	—	—	—
7 ($L = 20\frac{1}{2}$)	A325 F1852	N	STD	59.7	89.5	74.2	111	74.2	111	74.2	111	—	—	—	—
			SSLT	59.0	88.5	73.7	111	74.2	111	74.2	111	—	—	—	—
		X	STD	59.7	89.5	74.6	112	89.5	134	92.8	139	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	92.8	139	—	—	—	—
	A490	N	STD	59.7	89.5	74.6	112	89.5	134	92.8	139	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	92.8	139	—	—	—	—
		X	STD	59.7	89.5	74.6	112	89.5	134	104	157	—	—	—	—
			SSLT	59.0	88.5	73.7	111	88.5	133	103	155	—	—	—	—
6 ($L = 17\frac{1}{2}$)	A325 F1852	N	STD	51.6	77.4	63.6	95.4	63.6	95.4	63.6	95.4	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	63.6	95.4	63.6	95.4	—	—	—	—
		X	STD	51.6	77.4	64.5	96.7	77.4	116	79.5	119	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	79.5	119	—	—	—	—
	A490	N	STD	51.6	77.4	64.5	96.7	77.4	116	79.5	119	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	79.5	119	—	—	—	—
		X	STD	51.6	77.4	64.5	96.7	77.4	116	90.3	135	—	—	—	—
			SSLT	50.9	76.3	63.6	95.4	76.3	115	89.1	134	—	—	—	—
5 ($L = 14\frac{1}{2}$)	A325 F1852	N	STD	43.5	65.2	53.0	79.5	53.0	79.5	53.0	79.5	—	—	—	—
			SSLT	42.8	64.2	53.0	79.5	53.0	79.5	53.0	79.5	—	—	—	—
		X	STD	43.5	65.2	54.3	81.5	65.2	97.8	66.3	99.4	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	66.3	99.4	—	—	—	—
	A490	N	STD	43.5	65.2	54.3	81.5	65.2	97.8	66.3	99.4	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	66.3	99.4	—	—	—	—
		X	STD	43.5	65.2	54.3	81.5	65.2	97.8	76.1	114	—	—	—	—
			SSLT	42.8	64.2	53.5	80.2	64.2	96.3	74.9	112	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes

SSLT = Short-slotted holes transverse to direction of load

– indicates that the plate thickness is greater than $d_b/2 + {}^1/_{16}$ in.

Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included

X = Threads Excluded

Plate
F_y = 36 ksi

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

3/4-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
4 ($L = 11\tfrac{1}{2}$)	A325 F1852	N	STD	34.8	52.2	42.4	63.6	42.4	63.6	42.4	63.6	—	—	—	—
			SSLT	34.7	52.0	42.4	63.6	42.4	63.6	42.4	63.6	—	—	—	—
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	53.0	79.5	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	53.0	79.5	—	—	—	—
	A490	N	STD	34.8	52.2	43.5	65.3	52.2	78.3	53.0	79.5	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	53.0	79.5	—	—	—	—
		X	STD	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	—	—	—	—
			SSLT	34.7	52.0	43.4	65.1	52.0	78.1	60.7	91.1	—	—	—	—
3 ($L = 8\tfrac{1}{2}$)	A325 F1852	N	STD	25.6	38.3	31.8	47.7	31.8	47.7	31.8	47.7	—	—	—	—
			SSLT	25.6	38.3	31.8	47.7	31.8	47.7	31.8	47.7	—	—	—	—
		X	STD	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
	A490	N	STD	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	39.8	59.6	—	—	—	—
		X	STD	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	—	—	—	—
			SSLT	25.6	38.3	31.9	47.9	38.3	57.5	44.7	67.1	—	—	—	—
2 ($L = 5\tfrac{1}{2}$)	A325 F1852	N	STD	16.3	24.5	20.4	30.6	21.2	31.8	21.2	31.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	21.2	31.8	21.2	31.8	—	—	—	—
		X	STD	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
	A490	N	STD	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	26.5	39.8	—	—	—	—
		X	STD	16.3	24.5	20.4	30.6	24.5	36.7	28.5	42.8	—	—	—	—
			SSLT	16.3	24.5	20.4	30.6	24.5	36.7	28.5	42.8	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + {}^1/_{16}$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

7/8-in. diameter bolts

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 36 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (*L* = 36)	A325 F1852	N	STD	102	153	128	192	153	230	161	242	161	242	—	—
			SSLT	102	152	127	190	152	228	173	260	173	260	—	—
		X	STD	102	153	128	192	153	230	179	268	201	302	—	—
			SSLT	102	152	127	190	152	228	178	267	203	305	—	—
	A490	N	STD	102	153	128	192	153	230	179	268	201	302	—	—
			SSLT	102	152	127	190	152	228	178	267	203	305	—	—
		X	STD	102	153	128	192	153	230	179	268	204	307	—	—
			SSLT	102	152	127	190	152	228	178	267	203	305	—	—
11 (*L* = 33)	A325 F1852	N	STD	94.1	141	118	176	141	212	151	226	151	226	—	—
			SSLT	93.4	140	117	175	140	210	159	238	159	238	—	—
		X	STD	94.1	141	118	176	141	212	165	247	188	282	—	—
			SSLT	93.4	140	117	175	140	210	164	245	187	280	—	—
	A490	N	STD	94.1	141	118	176	141	212	165	247	188	282	—	—
			SSLT	93.4	140	117	175	140	210	164	245	187	280	—	—
		X	STD	94.1	141	118	176	141	212	165	247	188	282	—	—
			SSLT	93.4	140	117	175	140	210	164	245	187	280	—	—
10 (*L* = 30)	A325 F1852	N	STD	86.0	129	108	161	129	194	141	211	141	211	—	—
			SSLT	85.3	128	107	160	128	192	144	216	144	216	—	—
		X	STD	86.0	129	108	161	129	194	151	226	172	258	—	—
			SSLT	85.3	128	107	160	128	192	149	224	171	256	—	—
	A490	N	STD	86.0	129	108	161	129	194	151	226	172	258	—	—
			SSLT	85.3	128	107	160	128	192	149	224	171	256	—	—
		X	STD	86.0	129	108	161	129	194	151	226	172	258	—	—
			SSLT	85.3	128	107	160	128	192	149	224	171	256	—	—
9 (*L* = 27)	A325 F1852	N	STD	77.9	117	97.4	146	117	175	130	195	130	195	—	—
			SSLT	77.2	116	96.5	145	116	174	130	195	130	195	—	—
		X	STD	77.9	117	97.4	146	117	175	136	205	156	234	—	—
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	—	—
	A490	N	STD	77.9	117	97.4	146	117	175	136	205	156	234	—	—
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	—	—
		X	STD	77.9	117	97.4	146	117	175	136	205	156	234	—	—
			SSLT	77.2	116	96.5	145	116	174	135	203	154	232	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes

SSLT = Short-slotted holes transverse to direction of load

– indicates that the plate thickness is greater than $d_b/2 + ^1/_{16}$ in.

Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included

X = Threads Excluded

Plate
F_y = 36 ksi

Table 10-9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

7/8-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
8 (L = 24)	A325 F1852	N	STD	69.6	104	87.0	131	104	157	115	173	115	173	—	—
			SSLT	69.1	104	86.4	130	104	156	115	173	115	173	—	—
		X	STD	69.6	104	87.0	131	104	157	122	183	139	209	—	—
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	—	—
	A490	N	STD	69.6	104	87.0	131	104	157	122	183	139	209	—	—
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	—	—
		X	STD	69.6	104	87.0	131	104	157	122	183	139	209	—	—
			SSLT	69.1	104	86.4	130	104	156	121	181	138	207	—	—
7 (L = 21)	A325 F1852	N	STD/ SSLT	60.9	91.4	76.1	114	91.4	137	101	152	101	152	—	—
		X		60.9	91.4	76.1	114	91.4	137	107	160	122	183	—	—
	A490	N		60.9	91.4	76.1	114	91.4	137	107	160	122	183	—	—
		X		60.9	91.4	76.1	114	91.4	137	107	160	122	183	—	—
6 (L = 18)	A325 F1852	N	STD/ SSLT	52.2	78.3	65.3	97.9	78.3	117	86.6	130	86.6	130	—	—
		X		52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	—	—
	A490	N		52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	—	—
		X		52.2	78.3	65.3	97.9	78.3	117	91.4	137	104	157	—	—
5 (L = 15)	A325 F1852	N	STD/ SSLT	43.5	65.3	54.4	81.6	65.3	97.9	72.2	108	72.2	108	—	—
		X		43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	—	—
	A490	N		43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	—	—
		X		43.5	65.3	54.4	81.6	65.3	97.9	76.1	114	87.0	131	—	—
4 (L = 12)	A325 F1852	N	STD/ SSLT	34.8	52.2	43.5	65.3	52.2	78.3	57.7	86.6	57.7	86.6	—	—
		X		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	—	—
	A490	N		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	—	—
		X		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	—	—
3 (L = 9)	A325 F1852	N	STD/ SSLT	26.1	39.1	32.6	48.9	39.1	58.7	43.3	64.9	43.3	64.9	—	—
		X		26.1	39.1	32.6	48.9	39.1	58.7	45.7	68.5	52.2	78.3	—	—
	A490	N		26.1	39.1	32.6	48.9	39.1	58.7	45.7	68.5	52.2	78.3	—	—
		X		26.1	39.1	32.6	48.9	39.1	58.7	45.7	68.5	52.2	78.3	—	—
2 (L = 6)	A325 F1852	N	STD/ SSLT	17.4	26.1	21.8	32.6	26.1	39.1	28.9	43.3	28.9	43.3	—	—
		X		17.4	26.1	21.8	32.6	26.1	39.1	30.4	45.7	34.8	52.2	—	—
	A490	N		17.4	26.1	21.8	32.6	26.1	39.1	30.4	45.7	34.8	52.2	—	—
		X		17.4	26.1	21.8	32.6	26.1	39.1	30.4	45.7	34.8	52.2	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + {}^1/_{16}$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

1-in.
diameter
bolts

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate
F_y = 36 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (L = 36 1/2)	A325 F1852	N	STD	100	150	125	188	150	225	175	263	200	300	210	316
			SSLT	100	150	125	188	150	225	175	263	200	300	225	338
		X	STD	100	150	125	188	150	225	175	263	200	300	225	338
			SSLT	100	150	125	188	150	225	175	263	200	300	225	338
	A490	N	STD	100	150	125	188	150	225	175	263	200	300	225	338
			SSLT	100	150	125	188	150	225	175	263	200	300	225	338
		X	STD	100	150	125	188	150	225	175	263	200	300	225	338
			SSLT	100	150	125	188	150	225	175	263	200	300	225	338
11 (L = 33 1/2)	A325 F1852	N	STD	91.9	138	115	172	138	207	161	241	184	276	197	295
			SSLT	91.9	138	115	172	138	207	161	241	184	276	207	310
		X	STD	91.9	138	115	172	138	207	161	241	184	276	207	310
			SSLT	91.9	138	115	172	138	207	161	241	184	276	207	310
	A490	N	STD	91.9	138	115	172	138	207	161	241	184	276	207	310
			SSLT	91.9	138	115	172	138	207	161	241	184	276	207	310
		X	STD	91.9	138	115	172	138	207	161	241	184	276	207	310
			SSLT	91.9	138	115	172	138	207	161	241	184	276	207	310
10 (L = 30 1/2)	A325 F1852	N	STD	83.7	126	105	157	126	188	147	220	167	251	184	275
			SSLT	83.7	126	105	157	126	188	147	220	167	251	188	283
		X	STD	83.7	126	105	157	126	188	147	220	167	251	188	283
			SSLT	83.7	126	105	157	126	188	147	220	167	251	188	283
	A490	N	STD	83.7	126	105	157	126	188	147	220	167	251	188	283
			SSLT	83.7	126	105	157	126	188	147	220	167	251	188	283
		X	STD	83.7	126	105	157	126	188	147	220	167	251	188	283
			SSLT	83.7	126	105	157	126	188	147	220	167	251	188	283
9 (L = 27 1/2)	A325 F1852	N	STD/ SSLT	75.6	113	94.5	142	113	170	132	198	151	227	170	254
		X		75.6	113	94.5	142	113	170	132	198	151	227	170	255
	A490	N		75.6	113	94.5	142	113	170	132	198	151	227	170	255
		X		75.6	113	94.5	142	113	170	132	198	151	227	170	255
8 (L = 24 1/2)	A325 F1852	N	STD/ SSLT	67.4	101	84.3	126	101	152	118	177	135	202	151	226
		X		67.4	101	84.3	126	101	152	118	177	135	202	152	228
	A490	N		67.4	101	84.3	126	101	152	118	177	135	202	152	228
		X		67.4	101	84.3	126	101	152	118	177	135	202	152	228
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Plate
$F_y = 36$ ksi

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

1-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 ($L = 21\frac{1}{2}$)	A325	N	STD/ SSLT	59.3	88.9	74.1	111	88.9	133	104	156	119	178	132	198
	F1852	X		59.3	88.9	74.1	111	88.9	133	104	156	119	178	133	200
	A490	N		59.3	88.9	74.1	111	88.9	133	104	156	119	178	133	200
		X		59.3	88.9	74.1	111	88.9	133	104	156	119	178	133	200
6 ($L = 18\frac{1}{2}$)	A325	N	STD/ SSLT	51.1	76.7	63.9	95.8	76.7	115	89.4	134	102	153	113	170
	F1852	X		51.1	76.7	63.9	95.8	76.7	115	89.4	134	102	153	115	173
	A490	N		51.1	76.7	63.9	95.8	76.7	115	89.4	134	102	153	115	173
		X		51.1	76.7	63.9	95.8	76.7	115	89.4	134	102	153	115	173
5 ($L = 15\frac{1}{2}$)	A325	N	STD/ SSLT	43.0	64.4	53.7	80.5	64.4	96.7	75.2	113	85.9	129	94.2	141
	F1852	X		43.0	64.4	53.7	80.5	64.4	96.7	75.2	113	85.9	129	96.7	145
	A490	N		43.0	64.4	53.7	80.5	64.4	96.7	75.2	113	85.9	129	96.7	145
		X		43.0	64.4	53.7	80.5	64.4	96.7	75.2	113	85.9	129	96.7	145
4 ($L = 12\frac{1}{2}$)	A325	N	STD/ SSLT	34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	75.4	113
	F1852	X		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117
	A490	N		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117
		X		34.8	52.2	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117
3 ($L = 9\frac{1}{2}$)	A325	N	STD/ SSLT	26.6	40.0	33.3	50.0	40.0	59.9	46.6	69.9	53.3	79.9	56.5	84.8
	F1852	X		26.6	40.0	33.3	50.0	40.0	59.9	46.6	69.9	53.3	79.9	59.9	89.9
	A490	N		26.6	40.0	33.3	50.0	40.0	59.9	46.6	69.9	53.3	79.9	59.9	89.9
		X		26.6	40.0	33.3	50.0	40.0	59.9	46.6	69.9	53.3	79.9	59.9	89.9
2 ($L = 6\frac{1}{2}$)	A325	N	STD/ SSLT	18.5	27.7	23.1	34.7	27.7	41.6	32.4	48.5	37.0	55.5	37.7	56.5
	F1852	X		18.5	27.7	23.1	34.7	27.7	41.6	32.4	48.5	37.0	55.5	41.6	62.4
	A490	N		18.5	27.7	23.1	34.7	27.7	41.6	32.4	48.5	37.0	55.5	41.6	62.4
		X		18.5	27.7	23.1	34.7	27.7	41.6	32.4	48.5	37.0	55.5	41.6	62.4
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

1 1/8-in. diameter bolts

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 36 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (*L* = 37)	A325 F1852	N	STD	120	179	144	215	167	251	191	287	215	323	239	359
			SSLT	120	179	144	215	167	251	191	287	215	323	239	359
		X	STD	120	179	144	215	167	251	191	287	215	323	239	359
			SSLT	120	179	144	215	167	251	191	287	215	323	239	359
	A490	N	STD	120	179	144	215	167	251	191	287	215	323	239	359
			SSLT	120	179	144	215	167	251	191	287	215	323	239	359
		X	STD	120	179	144	215	167	251	191	287	215	323	239	359
			SSLT	120	179	144	215	167	251	191	287	215	323	239	359
11 (*L* = 34)	A325 F1852	N	STD	110	165	132	198	154	231	176	264	198	297	220	330
			SSLT	110	165	132	198	154	231	176	264	198	297	220	330
		X	STD	110	165	132	198	154	231	176	264	198	297	220	330
			SSLT	110	165	132	198	154	231	176	264	198	297	220	330
	A490	N	STD	110	165	132	198	154	231	176	264	198	297	220	330
			SSLT	110	165	132	198	154	231	176	264	198	297	220	330
		X	STD	110	165	132	198	154	231	176	264	198	297	220	330
			SSLT	110	165	132	198	154	231	176	264	198	297	220	330
10 (*L* = 31)	A325 F1852	N	STD	101	151	121	181	141	211	161	241	181	272	201	302
			SSLT	101	151	121	181	141	211	161	241	181	272	201	302
		X	STD	101	151	121	181	141	211	161	241	181	272	201	302
			SSLT	101	151	121	181	141	211	161	241	181	272	201	302
	A490	N	STD	101	151	121	181	141	211	161	241	181	272	201	302
			SSLT	101	151	121	181	141	211	161	241	181	272	201	302
		X	STD	101	151	121	181	141	211	161	241	181	272	201	302
			SSLT	101	151	121	181	141	211	161	241	181	272	201	302
9 (*L* = 28)	A325 F1852	N	STD/ SSLT	91.1	137	109	164	128	191	146	219	164	246	182	273
		X		91.1	137	109	164	128	191	146	219	164	246	182	273
	A490	N		91.1	137	109	164	128	191	146	219	164	246	182	273
		X		91.1	137	109	164	128	191	146	219	164	246	182	273
8 (*L* = 25)	A325 F1852	N	STD/ SSLT	81.6	122	97.9	147	114	171	131	196	147	220	163	245
		X		81.6	122	97.9	147	114	171	131	196	147	220	163	245
	A490	N		81.6	122	97.9	147	114	171	131	196	147	220	163	245
		X		81.6	122	97.9	147	114	171	131	196	147	220	163	245
Weld Size				1/4		1/4		5/16		5/16		3/8		7/16	

STD = Standard Holes

SSLT = Short-slotted holes transverse to direction of load

STD/SSLT = Standard holes or short-slotted holes transverse to direction of load

Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included

X = Threads Excluded

Plate
F_y = 36 ksi

Table 10–9a (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

1 1/8-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (L = 22)	A325	N	STD/ SSLT	72.0	108	86.5	130	101	151	115	173	130	195	144	216
	F1852	X		72.0	108	86.5	130	101	151	115	173	130	195	144	216
	A490	N		72.0	108	86.5	130	101	151	115	173	130	195	144	216
		X		72.0	108	86.5	130	101	151	115	173	130	195	144	216
6 (L = 19)	A325	N	STD/ SSLT	62.5	93.8	75.0	113	87.5	131	100	150	113	169	125	188
	F1852	X		62.5	93.8	75.0	113	87.5	131	100	150	113	169	125	188
	A490	N		62.5	93.8	75.0	113	87.5	131	100	150	113	169	125	188
		X		62.5	93.8	75.0	113	87.5	131	100	150	113	169	125	188
5 (L = 16)	A325	N	STD/ SSLT	53.0	79.5	63.6	95.4	74.2	111	84.8	127	95.4	143	106	159
	F1852	X		53.0	79.5	63.6	95.4	74.2	111	84.8	127	95.4	143	106	159
	A490	N		53.0	79.5	63.6	95.4	74.2	111	84.8	127	95.4	143	106	159
		X		53.0	79.5	63.6	95.4	74.2	111	84.8	127	95.4	143	106	159
4 (L = 13)	A325	N	STD/ SSLT	43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117	87.0	131
	F1852	X		43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117	87.0	131
	A490	N		43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117	87.0	131
		X		43.5	65.3	52.2	78.3	60.9	91.4	69.6	104	78.3	117	87.0	131
3 (L = 10)	A325	N	STD/ SSLT	34.0	51.0	40.8	61.2	47.6	71.4	54.4	81.6	61.2	91.8	68.0	102
	F1852	X		34.0	51.0	40.8	61.2	47.6	71.4	54.4	81.6	61.2	91.8	68.0	102
	A490	N		34.0	51.0	40.8	61.2	47.6	71.4	54.4	81.6	61.2	91.8	68.0	102
		X		34.0	51.0	40.8	61.2	47.6	71.4	54.4	81.6	61.2	91.8	68.0	102
2 (L = 7)	A325	N	STD/ SSLT	24.5	36.7	29.4	44.0	34.3	51.4	39.1	58.7	44.0	66.1	47.7	71.6
	F1852	X		24.5	36.7	29.4	44.0	34.3	51.4	39.1	58.7	44.0	66.1	48.9	73.4
	A490	N		24.5	36.7	29.4	44.0	34.3	51.4	39.1	58.7	44.0	66.1	48.9	73.4
		X		24.5	36.7	29.4	44.0	34.3	51.4	39.1	58.7	44.0	66.1	48.9	73.4
Weld Size				1/4		1/4		5/16		5/16		3/8		7/16	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

3/4-in. diameter bolts

Table 10–9b
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 50 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 ($L = 35\frac{1}{2}$)	A325 F1852	N	STD	118	178	118	178	118	178	118	178	—	—	—	—
			SSLT	122	183	127	191	127	191	127	191	—	—	—	—
		X	STD	122	183	148	222	148	222	148	222	—	—	—	—
			SSLT	122	183	152	229	159	239	159	239	—	—	—	—
	A490	N	STD	122	183	148	222	148	222	148	222	—	—	—	—
			SSLT	122	183	152	229	159	239	159	239	—	—	—	—
		X	STD	122	183	152	229	183	274	185	277	—	—	—	—
			SSLT	122	183	152	229	183	274	199	298	—	—	—	—
11 ($L = 32\frac{1}{2}$)	A325 F1852	N	STD	111	166	111	166	111	166	111	166	—	—	—	—
			SSLT	112	167	117	175	117	175	117	175	—	—	—	—
		X	STD	112	167	139	208	139	208	139	208	—	—	—	—
			SSLT	112	167	139	209	146	219	146	219	—	—	—	—
	A490	N	STD	112	167	139	208	139	208	139	208	—	—	—	—
			SSLT	112	167	139	209	146	219	146	219	—	—	—	—
		X	STD	112	167	139	209	167	251	173	260	—	—	—	—
			SSLT	112	167	139	209	167	251	182	273	—	—	—	—
10 ($L = 29\frac{1}{2}$)	A325 F1852	N	STD	101	152	103	155	103	155	103	155	—	—	—	—
			SSLT	101	152	106	159	106	159	106	159	—	—	—	—
		X	STD	101	152	126	190	129	194	129	194	—	—	—	—
			SSLT	101	152	126	190	133	199	133	199	—	—	—	—
	A490	N	STD	101	152	126	190	129	194	129	194	—	—	—	—
			SSLT	101	152	126	190	133	199	133	199	—	—	—	—
		X	STD	101	152	126	190	152	228	161	242	—	—	—	—
			SSLT	101	152	126	190	152	228	166	249	—	—	—	—
9 ($L = 26\frac{1}{2}$)	A325 F1852	N	STD/ SSLT	90.8	136	95.4	143	95.4	143	95.4	143	—	—	—	—
		X		90.8	136	113	170	119	179	119	179	—	—	—	—
	A490	N		90.8	136	113	170	119	179	119	179	—	—	—	—
		X		90.8	136	113	170	136	204	149	224	—	—	—	—
8 ($L = 23\frac{1}{2}$)	A325 F1852	N	STD/ SSLT	80.4	121	84.8	127	84.8	127	84.8	127	—	—	—	—
		X		80.4	121	101	151	106	159	106	159	—	—	—	—
	A490	N		80.4	121	101	151	106	159	106	159	—	—	—	—
		X		80.4	121	101	151	121	181	133	199	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes

SSLT = Short-slotted holes transverse to direction of load

STD/SSLT = Standard holes or short-slotted holes transverse to direction of load

– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.

Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included

X = Threads Excluded

Plate
F_y = 50 ksi

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

3/4-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 ($L = 20\frac{1}{2}$)	A325 F1852	N	STD/ SSLT	70.1	105	74.2	111	74.2	111	74.2	111	—	—	—	—
		X		70.1	105	87.6	131	92.8	139	92.8	139	—	—	—	—
	A490	N		70.1	105	87.6	131	92.8	139	92.8	139	—	—	—	—
		X		70.1	105	87.6	131	105	158	116	174	—	—	—	—
6 ($L = 17\frac{1}{2}$)	A325 F1852	N	STD/ SSLT	59.7	89.6	63.6	95.4	63.6	95.4	63.6	95.4	—	—	—	—
		X		59.7	89.6	74.6	112	79.5	119	79.5	119	—	—	—	—
	A490	N		59.7	89.6	74.6	112	79.5	119	79.5	119	—	—	—	—
		X		59.7	89.6	74.6	112	89.6	134	99.4	149	—	—	—	—
5 ($L = 14\frac{1}{2}$)	A325 F1852	N	STD/ SSLT	49.4	74.0	53.0	79.5	53.0	79.5	53.0	79.5	—	—	—	—
		X		49.4	74.0	61.7	92.5	66.3	99.4	66.3	99.4	—	—	—	—
	A490	N		49.4	74.0	61.7	92.5	66.3	99.4	66.3	99.4	—	—	—	—
		X		49.4	74.0	61.7	92.5	74.0	111	82.8	124	—	—	—	—
4 ($L = 11\frac{1}{2}$)	A325 F1852	N	STD/ SSLT	39.0	58.5	42.4	63.6	42.4	63.6	42.4	63.6	—	—	—	—
		X		39.0	58.5	48.8	73.1	53.0	79.5	53.0	79.5	—	—	—	—
	A490	N		39.0	58.5	48.8	73.1	53.0	79.5	53.0	79.5	—	—	—	—
		X		39.0	58.5	48.8	73.1	58.5	87.8	66.3	99.4	—	—	—	—
3 ($L = 8\frac{1}{2}$)	A325 F1852	N	STD/ SSLT	28.6	43.0	31.8	47.7	31.8	47.7	31.8	47.7	—	—	—	—
		X		28.6	43.0	35.8	53.7	39.8	59.6	39.8	59.6	—	—	—	—
	A490	N		28.6	43.0	35.8	53.7	39.8	59.6	39.8	59.6	—	—	—	—
		X		28.6	43.0	35.8	53.7	43.0	64.4	49.7	74.6	—	—	—	—
2 ($L = 5\frac{1}{2}$)	A325 F1852	N	STD/ SSLT	18.3	27.4	21.2	31.8	21.2	31.8	21.2	31.8	—	—	—	—
		X		18.3	27.4	22.9	34.3	26.5	39.8	26.5	39.8	—	—	—	—
	A490	N		18.3	27.4	22.9	34.3	26.5	39.8	26.5	39.8	—	—	—	—
		X		18.3	27.4	22.9	34.3	27.4	41.1	32.0	48.0	—	—	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + {}^1/_{16}$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

7/8-in. diameter bolts

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 50 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (*L* = 36)	A325 F1852	N	STD	117	176	146	219	161	242	161	242	161	242	—	—
			SSLT	117	176	146	219	173	260	173	260	173	260	—	—
		X	STD	117	176	146	219	176	263	201	302	201	302	—	—
			SSLT	117	176	146	219	176	263	205	307	216	325	—	—
	A490	N	STD	117	176	146	219	176	263	201	302	201	302	—	—
			SSLT	117	176	146	219	176	263	205	307	216	325	—	—
		X	STD	117	176	146	219	176	263	205	307	234	351	—	—
			SSLT	117	176	146	219	176	263	205	307	234	351	—	—
11 (*L* = 33)	A325 F1852	N	STD	107	161	134	201	151	226	151	226	151	226	—	—
			SSLT	107	161	134	201	159	238	159	238	159	238	—	—
		X	STD	107	161	134	201	161	241	188	282	189	283	—	—
			SSLT	107	161	134	201	161	241	188	282	198	298	—	—
	A490	N	STD	107	161	134	201	161	241	188	282	189	283	—	—
			SSLT	107	161	134	201	161	241	188	282	198	298	—	—
		X	STD	107	161	134	201	161	241	188	282	215	322	—	—
			SSLT	107	161	134	201	161	241	188	282	215	322	—	—
10 (*L* = 30)	A325 F1852	N	STD	97.5	146	122	183	141	211	141	211	141	211	—	—
			SSLT	97.5	146	122	183	144	216	144	216	144	216	—	—
		X	STD	97.5	146	122	183	146	219	171	256	176	263	—	—
			SSLT	97.5	146	122	183	146	219	171	256	180	271	—	—
	A490	N	STD	97.5	146	122	183	146	219	171	256	176	263	—	—
			SSLT	97.5	146	122	183	146	219	171	256	180	271	—	—
		X	STD	97.5	146	122	183	146	219	171	256	195	293	—	—
			SSLT	97.5	146	122	183	146	219	171	256	195	293	—	—
9 (*L* = 27)	A325 F1852	N	STD/ SSLT	87.8	132	110	165	130	195	130	195	130	195	—	—
		X		87.8	132	110	165	132	197	154	230	162	244	—	—
	A490	N		87.8	132	110	165	132	197	154	230	162	244	—	—
		X		87.8	132	110	165	132	197	154	230	176	263	—	—
8 (*L* = 24)	A325 F1852	N	STD/ SSLT	78.0	117	97.5	146	115	173	115	173	115	173	—	—
		X		78.0	117	97.5	146	117	176	137	205	144	216	—	—
	A490	N		78.0	117	97.5	146	117	176	137	205	144	216	—	—
		X		78.0	117	97.5	146	117	176	137	205	156	234	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes

SSLT = Short-slotted holes transverse to direction of load

STD/SSLT = Standard holes or short-slotted holes transverse to direction of load

– indicates that the plate thickness is greater than $d_b/2 + {}^1/_{16}$ in.

Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included

X = Threads Excluded

Plate
F_y = 50 ksi

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

7/8-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (*L* = 21)	A325 F1852	N	STD/ SSLT	68.3	102	85.3	128	101	152	101	152	101	152	—	—
		X		68.3	102	85.3	128	102	154	119	179	126	189	—	—
	A490	N		68.3	102	85.3	128	102	154	119	179	126	189	—	—
		X		68.3	102	85.3	128	102	154	119	179	137	205	—	—
6 (*L* = 18)	A325 F1852	N	STD/ SSLT	58.5	87.8	73.1	110	86.6	130	86.6	130	86.6	130	—	—
		X		58.5	87.8	73.1	110	87.8	132	102	154	108	162	—	—
	A490	N		58.5	87.8	73.1	110	87.8	132	102	154	108	162	—	—
		X		58.5	87.8	73.1	110	87.8	132	102	154	117	176	—	—
5 (*L* = 15)	A325 F1852	N	STD/ SSLT	48.8	73.1	60.9	91.4	72.2	108	72.2	108	72.2	108	—	—
		X		48.8	73.1	60.9	91.4	73.1	110	85.3	128	90.2	135	—	—
	A490	N		48.8	73.1	60.9	91.4	73.1	110	85.3	128	90.2	135	—	—
		X		48.8	73.1	60.9	91.4	73.1	110	85.3	128	97.5	146	—	—
4 (*L* = 12)	A325 F1852	N	STD/ SSLT	39.0	58.5	48.8	73.1	57.7	86.6	57.7	86.6	57.7	86.6	—	—
		X		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	72.2	108	—	—
	A490	N		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	72.2	108	—	—
		X		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	—	—
3 (*L* = 9)	A325 F1852	N	STD/ SSLT	29.3	43.9	36.6	54.8	43.3	64.9	43.3	64.9	43.3	64.9	—	—
		X		29.3	43.9	36.6	54.8	43.9	65.8	51.2	76.8	54.1	81.2	—	—
	A490	N		29.3	43.9	36.6	54.8	43.9	65.8	51.2	76.8	54.1	81.2	—	—
		X		29.3	43.9	36.6	54.8	43.9	65.8	51.2	76.8	58.5	87.8	—	—
2 (*L* = 6)	A325 F1852	N	STD/ SSLT	19.5	29.3	24.4	36.6	28.9	43.3	28.9	43.3	28.9	43.3	—	—
		X		19.5	29.3	24.4	36.6	29.3	43.9	34.1	51.2	36.1	54.1	—	—
	A490	N		19.5	29.3	24.4	36.6	29.3	43.9	34.1	51.2	36.1	54.1	—	—
		X		19.5	29.3	24.4	36.6	29.3	43.9	34.1	51.2	39.0	58.5	—	—
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
– indicates that the plate thickness is greater than $d_b/2 + 1/16$ in.
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

1-in. diameter bolts

Table 10–9b (continued)

Single-Plate Connections

Bolt, Weld, and Single-Plate Available Strengths, kips

Plate
F_y = 50 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				$^{1}/_{4}$		$^{5}/_{16}$		$^{3}/_{8}$		$^{7}/_{16}$		$^{1}/_{2}$		$^{9}/_{16}$	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (L = 36$^{1}/_{2}$)	A325 F1852	N	STD	112	168	140	210	168	252	196	294	210	316	210	316
			SSLT	112	168	140	210	168	252	196	294	224	336	226	339
		X	STD	112	168	140	210	168	252	196	294	224	336	252	378
			SSLT	112	168	140	210	168	252	196	294	224	336	252	378
	A490	N	STD	112	168	140	210	168	252	196	294	224	336	252	378
			SSLT	112	168	140	210	168	252	196	294	224	336	252	378
		X	STD	112	168	140	210	168	252	196	294	224	336	252	378
			SSLT	112	168	140	210	168	252	196	294	224	336	252	378
11 (L = 33$^{1}/_{2}$)	A325 F1852	N	STD	103	154	129	193	154	232	180	270	197	295	197	295
			SSLT	103	154	129	193	154	232	180	270	206	309	207	311
		X	STD	103	154	129	193	154	232	180	270	206	309	232	348
			SSLT	103	154	129	193	154	232	180	270	206	309	232	348
	A490	N	STD	103	154	129	193	154	232	180	270	206	309	232	348
			SSLT	103	154	129	193	154	232	180	270	206	309	232	348
		X	STD	103	154	129	193	154	232	180	270	206	309	232	348
			SSLT	103	154	129	193	154	232	180	270	206	309	232	348
10 (L = 30$^{1}/_{2}$)	A325 F1852	N	STD	93.8	141	117	176	141	211	164	246	184	275	184	275
			SSLT	93.8	141	117	176	141	211	164	246	188	282	188	283
		X	STD	93.8	141	117	176	141	211	164	246	188	282	211	317
			SSLT	93.8	141	117	176	141	211	164	246	188	282	211	317
	A490	N	STD	93.8	141	117	176	141	211	164	246	188	282	211	317
			SSLT	93.8	141	117	176	141	211	164	246	188	282	211	317
		X	STD	93.8	141	117	176	141	211	164	246	188	282	211	317
			SSLT	93.8	141	117	176	141	211	164	246	188	282	211	317
9 (L = 27$^{1}/_{2}$)	A325 F1852	N	STD/ SSLT	84.7	127	106	159	127	191	148	222	169	254	170	254
		X		84.7	127	106	159	127	191	148	222	169	254	191	286
	A490	N		84.7	127	106	159	127	191	148	222	169	254	191	286
		X		84.7	127	106	159	127	191	148	222	169	254	191	286
8 (L = 24$^{1}/_{2}$)	A325 F1852	N	STD/ SSLT	75.6	113	94.5	142	113	170	132	198	151	226	151	226
		X		75.6	113	94.5	142	113	170	132	198	151	227	170	255
	A490	N		75.6	113	94.5	142	113	170	132	198	151	227	170	255
		X		75.6	113	94.5	142	113	170	132	198	151	227	170	255
Weld Size				$^{3}/_{16}$		$^{1}/_{4}$		$^{1}/_{4}$		$^{5}/_{16}$		$^{5}/_{16}$		$^{3}/_{8}$	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Plate
F_y = 50 ksi

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

1-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				1/4		5/16		3/8		7/16		1/2		9/16	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (*L* = 21 1/2)	A325	N	STD/ SSLT	66.4	99.6	83.0	125	99.6	149	116	174	132	198	132	198
	F1852	X		66.4	99.6	83.0	125	99.6	149	116	174	133	199	149	224
	A490	N		66.4	99.6	83.0	125	99.6	149	116	174	133	199	149	224
		X		66.4	99.6	83.0	125	99.6	149	116	174	133	199	149	224
6 (*L* = 18 1/2)	A325	N	STD/ SSLT	57.3	85.9	71.6	107	85.9	129	100	150	113	170	113	170
	F1852	X		57.3	85.9	71.6	107	85.9	129	100	150	115	172	129	193
	A490	N		57.3	85.9	71.6	107	85.9	129	100	150	115	172	129	193
		X		57.3	85.9	71.6	107	85.9	129	100	150	115	172	129	193
5 (*L* = 15 1/2)	A325	N	STD/ SSLT	48.1	72.2	60.2	90.3	72.2	108	84.2	126	94.2	141	94.2	141
	F1852	X		48.1	72.2	60.2	90.3	72.2	108	84.2	126	96.3	144	108	162
	A490	N		48.1	72.2	60.2	90.3	72.2	108	84.2	126	96.3	144	108	162
		X		48.1	72.2	60.2	90.3	72.2	108	84.2	126	96.3	144	108	162
4 (*L* = 12 1/2)	A325	N	STD/ SSLT	39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	75.4	113	75.4	113
	F1852	X		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132
	A490	N		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132
		X		39.0	58.5	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132
3 (*L* = 9 1/2)	A325	N	STD/ SSLT	29.9	44.8	37.3	56.0	44.8	67.2	52.3	78.4	56.5	84.8	56.5	84.8
	F1852	X		29.9	44.8	37.3	56.0	44.8	67.2	52.3	78.4	59.7	89.6	67.2	101
	A490	N		29.9	44.8	37.3	56.0	44.8	67.2	52.3	78.4	59.7	89.6	67.2	101
		X		29.9	44.8	37.3	56.0	44.8	67.2	52.3	78.4	59.7	89.6	67.2	101
2 (*L* = 6 1/2)	A325	N	STD/ SSLT	20.7	31.1	25.9	38.8	31.1	46.6	36.3	54.4	37.7	56.5	37.7	56.5
	F1852	X		20.7	31.1	25.9	38.8	31.1	46.6	36.3	54.4	41.4	62.2	46.6	69.9
	A490	N		20.7	31.1	25.9	38.8	31.1	46.6	36.3	54.4	41.4	62.2	46.6	69.9
		X		20.7	31.1	25.9	38.8	31.1	46.6	36.3	54.4	41.4	62.2	46.6	69.9
Weld Size				3/16		1/4		1/4		5/16		5/16		3/8	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

1 1/8-in. diameter bolts

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

Plate F_y = 50 ksi

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
12 (*L* = 37)	A325 F1852	N	STD	134	201	161	241	188	282	215	322	241	362	266	399
			SSLT	134	201	161	241	188	282	215	322	241	362	268	402
		X	STD	134	201	161	241	188	282	215	322	241	362	268	402
			SSLT	134	201	161	241	188	282	215	322	241	362	268	402
	A490	N	STD	134	201	161	241	188	282	215	322	241	362	268	402
			SSLT	134	201	161	241	188	282	215	322	241	362	268	402
		X	STD	134	201	161	241	188	282	215	322	241	362	268	402
			SSLT	134	201	161	241	188	282	215	322	241	362	268	402
11 (*L* = 34)	A325 F1852	N	STD	123	185	148	222	173	259	197	296	222	333	247	370
			SSLT	123	185	148	222	173	259	197	296	222	333	247	370
		X	STD	123	185	148	222	173	259	197	296	222	333	247	370
			SSLT	123	185	148	222	173	259	197	296	222	333	247	370
	A490	N	STD	123	185	148	222	173	259	197	296	222	333	247	370
			SSLT	123	185	148	222	173	259	197	296	222	333	247	370
		X	STD	123	185	148	222	173	259	197	296	222	333	247	370
			SSLT	123	185	148	222	173	259	197	296	222	333	247	370
10 (*L* = 31)	A325 F1852	N	STD	113	169	135	203	158	237	180	271	203	304	225	338
			SSLT	113	169	135	203	158	237	180	271	203	304	225	338
		X	STD	113	169	135	203	158	237	180	271	203	304	225	338
			SSLT	113	169	135	203	158	237	180	271	203	304	225	338
	A490	N	STD	113	169	135	203	158	237	180	271	203	304	225	338
			SSLT	113	169	135	203	158	237	180	271	203	304	225	338
		X	STD	113	169	135	203	158	237	180	271	203	304	225	338
			SSLT	113	169	135	203	158	237	180	271	203	304	225	338
9 (*L* = 28)	A325 F1852	N	STD/ SSLT	102	153	122	184	143	214	163	245	184	276	204	306
		X		102	153	122	184	143	214	163	245	184	276	204	306
	A490	N		102	153	122	184	143	214	163	245	184	276	204	306
		X		102	153	122	184	143	214	163	245	184	276	204	306
8 (*L* = 25)	A325 F1852	N	STD/ SSLT	91.4	137	110	165	128	192	146	219	165	247	183	274
		X		91.4	137	110	165	128	192	146	219	165	247	183	274
	A490	N		91.4	137	110	165	128	192	146	219	165	247	183	274
		X		91.4	137	110	165	128	192	146	219	165	247	183	274
Weld Size				1/4		1/4		5/16		5/16		3/8		7/16	

STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

Plate
F_y = 50 ksi

Table 10–9b (continued)
Single-Plate Connections
Bolt, Weld, and Single-Plate Available Strengths, kips

1 1/8-in. diameter bolts

n	ASTM Desig.	Thread Cond.	Hole Type	Plate Thickness, in.											
				5/16		3/8		7/16		1/2		9/16		5/8	
				ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
7 (L = 22)	A325 F1852	N	STD/ SSLT	80.7	121	96.9	145	113	170	129	194	145	218	161	242
		X		80.7	121	96.9	145	113	170	129	194	145	218	161	242
	A490	N		80.7	121	96.9	145	113	170	129	194	145	218	161	242
		X		80.7	121	96.9	145	113	170	129	194	145	218	161	242
6 (L = 19)	A325 F1852	N	STD/ SSLT	70.1	105	84.1	126	98.1	147	112	168	126	189	140	210
		X		70.1	105	84.1	126	98.1	147	112	168	126	189	140	210
	A490	N		70.1	105	84.1	126	98.1	147	112	168	126	189	140	210
		X		70.1	105	84.1	126	98.1	147	112	168	126	189	140	210
5 (L = 16)	A325 F1852	N	STD/ SSLT	59.4	89.1	71.3	107	83.2	125	95.1	143	107	160	119	178
		X		59.4	89.1	71.3	107	83.2	125	95.1	143	107	160	119	178
	A490	N		59.4	89.1	71.3	107	83.2	125	95.1	143	107	160	119	178
		X		59.4	89.1	71.3	107	83.2	125	95.1	143	107	160	119	178
4 (L = 13)	A325 F1852	N	STD/ SSLT	48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132	95.4	143
		X		48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132	97.5	146
	A490	N		48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132	97.5	146
		X		48.8	73.1	58.5	87.8	68.3	102	78.0	117	87.8	132	97.5	146
3 (L = 10)	A325 F1852	N	STD/ SSLT	38.1	57.1	45.7	68.6	53.3	80.0	60.9	91.4	68.6	103	71.6	107
		X		38.1	57.1	45.7	68.6	53.3	80.0	60.9	91.4	68.6	103	76.2	114
	A490	N		38.1	57.1	45.7	68.6	53.3	80.0	60.9	91.4	68.6	103	76.2	114
		X		38.1	57.1	45.7	68.6	53.3	80.0	60.9	91.4	68.6	103	76.2	114
2 (L = 7)	A325 F1852	N	STD/ SSLT	27.4	41.1	32.9	49.4	38.4	57.6	43.9	65.8	47.7	71.6	47.7	71.6
		X		27.4	41.1	32.9	49.4	38.4	57.6	43.9	65.8	49.4	74.0	54.8	82.3
	A490	N		27.4	41.1	32.9	49.4	38.4	57.6	43.9	65.8	49.4	74.0	54.8	82.3
		X		27.4	41.1	32.9	49.4	38.4	57.6	43.9	65.8	49.4	74.0	54.8	82.3
Weld Size				1/4		1/4		5/16		5/16		3/8		7/16	

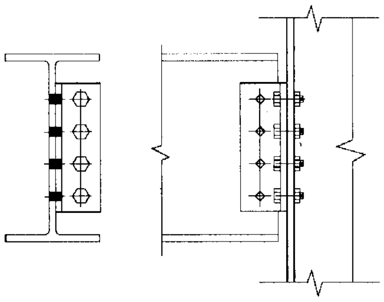
STD = Standard Holes
SSLT = Short-slotted holes transverse to direction of load
STD/SSLT = Standard holes or short-slotted holes transverse to direction of load
Tabulated values are grouped when available strength is independent of hole type.

N = Threads Included
X = Threads Excluded

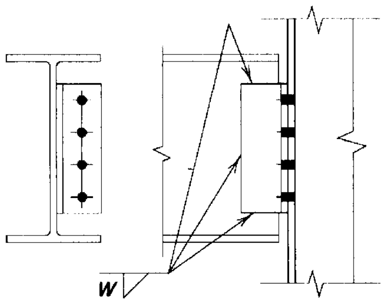
SINGLE-ANGLE CONNECTIONS

A single-angle connection is made with an angle on one side of the web of the beam to be supported, as illustrated in Figure 10–13. This angle is preferably shop-bolted or welded to the supporting member and field-bolted to the supported beam.

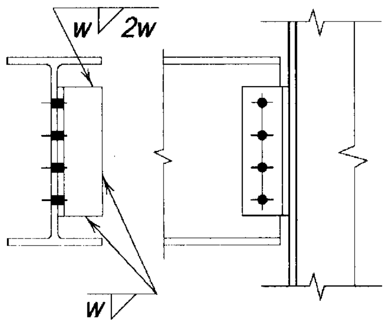
When the angle is welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10–13c, the weld is placed along the toe and across the bottom of the angle with a return at the top per AISC Specification Section J2.2b. Note that welding across the entire top of the angle must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for simple shear connections.



(a) All-bolted



(b) Bolted/welded, angle welded to supported beam



Note: weld return on top of angle per Specification Section J2.2b.

(c) Bolted/welded, angle welded to support

Figure 10–13. Single-angle connections.

Design Checks

The available strength of a single-angle connection is determined from the applicable limit states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

As illustrated in Figure 10–14, the effect of eccentricity should always be considered in the angle leg attached to the support. Additionally, eccentricity should be considered in the case of a double vertical row of bolts through the web of the supported beam or if the eccentricity exceeds 3 in. ($2^3/_4$-in. gage plus $^1/_4$-in. half web). Eccentricity should always be considered in the design of welds for single-angle connections.

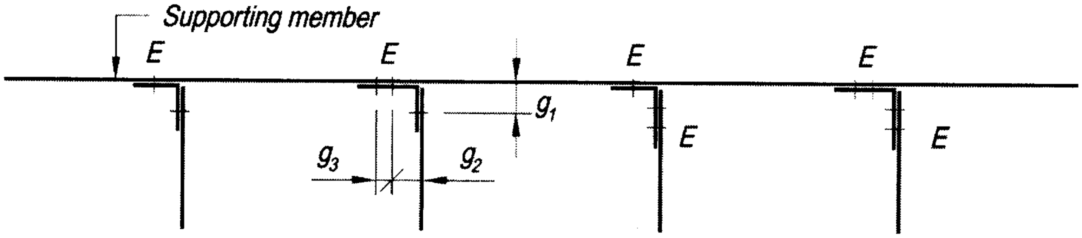
Recommended Angle Length and Thickness

To provide for stability during erection, it is recommended that the mimimum angle length be one-half the *T*-dimension of the beam to be supported. The maximum length of the connection angles must be compatible with the *T*-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam. Note that the angle may encroach upon the fillet(s) as given in Figure 10–3.

A minimum angle thickness of $^3/_8$-in. for $^3/_4$-in. and $^7/_8$-in. diameter bolts, and $^1/_2$-in. for 1-in. diameter bolts should be used. A 4×3 angle is normally selected for a single angle welded to the support with the 3-in. leg being the welded leg.

Shop and Field Practices

Single-angle connections may be readily made to the webs of supporting girders and to the flanges of supporting columns. When framing to a column flange, provision must be made for possible mill variation in the depth of the columns. Since the angle is usually shop-attached to the column flange, play in the open holes or horizontal slots in the angle leg may be used to provide the necessary adjustment to compensate for the mill variation. Attaching the angle to the column flange offers the advantage of side erection of the beam. The same is true for a girder web or truss support. Additionally, proper bay dimensions may be maintained without the need for shims. This advantage is lost in the case that the angle is shop-attached to the supported beam web.



E indicates that eccentricity must be considered in this leg.
Gages g_1, g_2, and g_3 are workable gages as shown in Table 1-7

Figure 10–14. Eccentricity in angles.

Table 10–10. All-Bolted Single-Angle Connections

Table 10–10 is a design aid for all-bolted single-angle connections. The tabulated eccentrically loaded bolt group coefficients, C, are useful in determining the available strength, ϕR_n or R_n/Ω, where

$$R_n = C \times r_n$$

$$\phi = 0.75 \qquad \Omega = 2.0$$

In the above equation,

C = coefficient from Table 10–10
r_n = the nominal strength of one bolt in shear or bearing, kips

Table 10–11. Bolted/Welded Single-Angle Connections

Table 10–11 is a design aid for bolted/welded single-angle connections. Electrode strength is assumed to be 70 ksi. In the rare case where a single-angle connection must be field-welded, erection bolts may be placed in the leg to be field-welded.

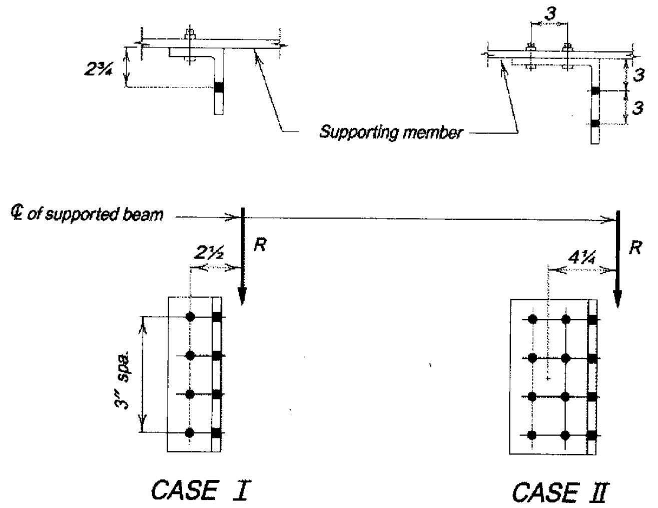
Weld available strengths are determined by the instantaneous center of rotation method using Table 8-11 with $\theta = 0°$. The tabulated values assume a half-web thickness of $^1/_4$ in. and may be used conservatively for lesser half-web thicknesses. For half-web thicknesses greater than $^1/_4$ in., the tabulated values should be reduced proportionally to eight percent at a half-web thickness of $^1/_2$ in. The tabulated minimum supporting flange or web thickness is the thickness that matches the strength of the support material to the strength of the weld material. In a manner similar to that illustrated previously for Table 10–2, the minimum material thickness (for one line of weld) may be calculated as:

$$t_{min} = \frac{3.09D}{F_u}$$

where D is the number of sixteenths in the weld size. When welds line up on opposite sides of the support, the minimum thickness is the sum of the thicknesses required for each weld. In either case, when less than the minimum material thickness is present, the tabulated weld available strength should be multiplied by the ratio of the thickness provided to the minimum thickness.

Table 10–10

All-Bolted Single-Angle Connections



Note: standard holes in support leg of angle

Eccentrically Loaded Bolt Group Coefficients, *C*

Number of Bolts in One Vertical Row, *n*	Case I	Case II
12	11.4	21.5
11	10.4	19.4
10	9.37	17.3
9	8.34	15.2
8	7.31	13.0
7	6.27	10.9
6	5.22	8.70
5	4.15	6.63
4	3.07	4.70
3	1.99	2.94
2	1.03	1.61
1	—	0.518

$\phi R_n = C \times \phi r_n$ or $R_n/\Omega = C \times r_n/\Omega$

where

C = coefficient from Table above

ϕr_n = design strength of one bolt in shear or bearing, kips/bolt

r_n/Ω = allowable strength of one bolt in shear or bearing, kips/bolt

Notes:

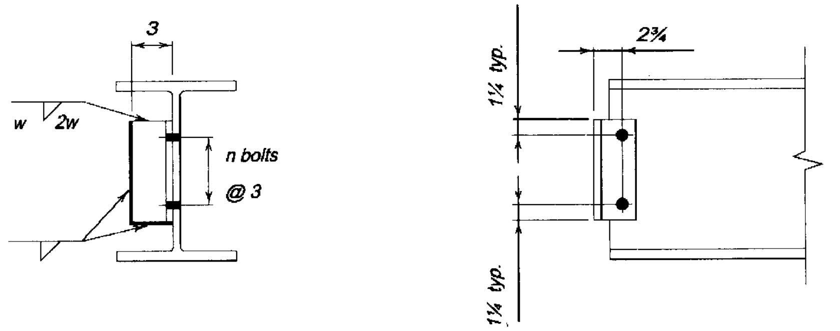
For eccentricities less than or equal to those shown above, tabulated values may be used.

For greater eccentricities, coefficient C should be recalculated from Part 7.

Connection may be bearing-type or slip-critical.

Table 10–11

Bolted/Welded Single-Angle Connections



Number of Bolts in One Vertical Row	Bolt and Angle Strength, kips				Angle Size (F_y = 36 ksi)	Angle Length, in.	Weld (70 ksi)			Minimum t_w of Supporting Member with Angles Both Sides of Web, in.
	3/4 in.		7/8 in.				Size, w, in.	Available Strength, kips		
	ASD	LRFD	ASD	LRFD				ASD	LRFD	
12	127	191	147	220	L4×3×3/8	35½	5/16	176	265	0.476
							1/4	141	212	0.381
							3/16	106	159	0.286
11	117	175	135	202		32½	5/16	164	246	0.476
							1/4	131	197	0.381
							3/16	98.6	148	0.286
10	106	159	123	184		29½	5/16	151	227	0.476
							1/4	121	181	0.381
							3/16	90.6	136	0.286
9	95.4	143	110	166		26½	5/16	137	206	0.476
							1/4	110	165	0.381
							3/16	82.3	123	0.286
8	84.8	127	98.3	147		23½	5/16	123	185	0.476
							1/4	98.7	148	0.381
							3/16	74	111	0.286
7	74.2	111	86.1	129		20½	5/16	109	164	0.476
							1/4	87.5	131	0.381
							3/16	65.6	98.4	0.286

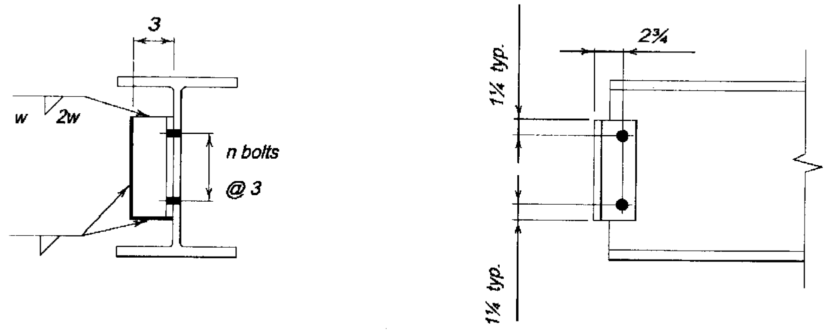
Notes:

Gage in angle leg attached to beam web as well as leg width may be decreased. 3-in. welded leg may not be increased or decreased.

Tabulated weld available strengths are based on a 1/4-in. half web for the supported member. Smaller half webs will result in these values being conservative. For half webs over 1/4 in., weld values must be reduced proportionally to 8% for a 1/2-in. half web or recalculated.

When the beam web thickness of the supporting member is less than the minimum and single-angle connections are back to back, either stagger the angles, or multiply the weld design strength by the ratio of the actual web thickness to the tabulated minimum thickness to determine the reduced weld design strength.

Table 10-11 (continued)
Bolted/Welded Single-Angle Connections



Number of Bolts in One Vertical Row	Bolt and Angle Strength, kips				Angle Size (F_y = 36 ksi)	Angle Length, in.	Weld (70 ksi)			Minimum t_w of Supporting Member with Angles Both Sides of Web, in.
	$^3/_4$ in.		$^7/_8$ in.				Size, w, in.	Available Strength, kips		
	ASD	LRFD	ASD	LRFD				ASD	LRFD	
6	63.6	95.4	74	111	L4×3×$^3/_8$	17$^1/_2$	$^5/_{16}$	94.3	142	0.476
							$^1/_4$	75.5	113	0.381
							$^3/_{16}$	56.6	84.9	0.286
5	53	79.5	61.8	92.7		14$^1/_2$	$^5/_{16}$	79	119	0.476
							$^1/_4$	63.2	94.8	0.381
							$^3/_{16}$	47.4	71.1	0.286
4	42.4	63.6	48.9	73.4		11$^1/_2$	$^5/_{16}$	62.8	94.3	0.476
							$^1/_4$	50.3	75.4	0.381
							$^3/_{16}$	37.7	56.6	0.286
3	31.8	47.7	35.9	53.8		8$^1/_2$	$^5/_{16}$	45.7	68.5	0.476
							$^1/_4$	36.5	54.8	0.381
							$^3/_{16}$	27.4	41.1	0.286
2	21.2	31.8	22.8	34.3		5$^1/_2$	$^5/_{16}$	28.1	42.2	0.476
							$^1/_4$	22.5	33.7	0.381
							$^3/_{16}$	16.9	25.3	0.286

Notes:

Gage in angle leg attached to beam web as well as leg width may be decreased. 3-in. welded leg may not be increased or decreased.

Tabulated weld available strengths are based on a $^1/_4$-in. half web for the supported member. Smaller half webs will result in these values being conservative. For half webs over $^1/_4$ in., weld values must be reduced proportionally to 8% for a $^1/_2$-in. half web or recalculated.

When the beam web thickness of the supporting member is less than the minimum and single-angle connections are back to back, either stagger the angles, or multiply the weld design strength by the ratio of the actual web thickness to the tabulated minimum thickness to determine the reduced weld design strength.

TEE CONNECTIONS

A tee connection is made with a structural tee, as illustrated in Figure 10–15. The tee is preferably shop-bolted or welded to the supporting member and field-bolted to the supported beam.

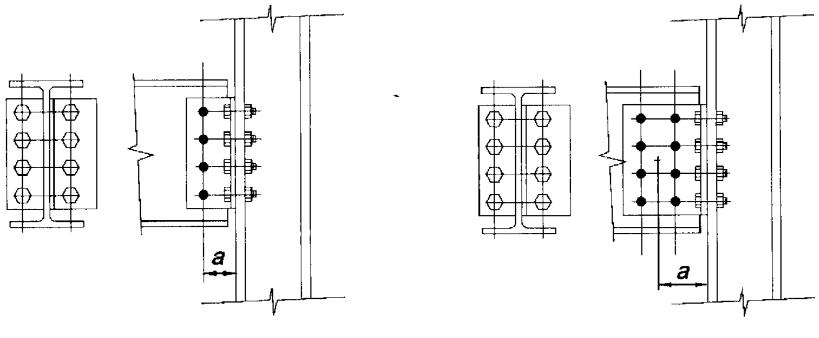
When the tee is welded to the support, adequate flexibility must be provided in the connection. As illustrated in Figure 10–15b, line welds are placed along the toes of the tee flange with a return at the top per AISC Specification Section J2.2b. Note that welding across the entire top of the tee must be avoided as it would inhibit the flexibility and, therefore, the necessary end rotation of the connection. The performance of the resulting connection would not be as intended for simple shear connections.

Design Checks

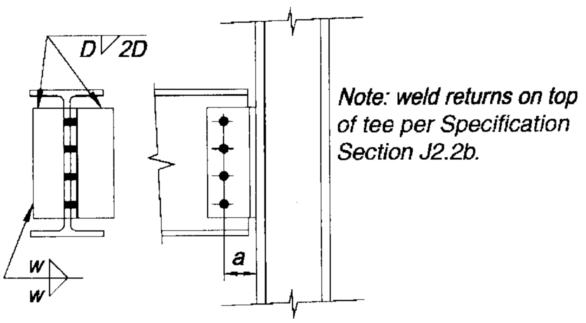
The available strength of a tee connection is determined from the applicable limit-states for bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Eccentricity must be considered when determining the available strength of tee connections. For a flexible support, the bolts or welds attaching the tee flange to the support must be designed for the shear, R_u or R_a. Also, the bolts through the tee stem must be designed for the shear and the eccentric moment, $R_u a$ or $R_a a$, where a is the distance from the face of the support to the centroid of the bolt group through the tee stem.

For a rigid support, the bolts or welds attaching the tee flange to the support must be designed for the shear and the eccentric moment; the bolts through the tee stem must be designed for the shear.



(a) All-bolted



Note: weld returns on top of tee per Specification Section J2.2b.

(b) Bolted/welded

Figure 10–15. Tee connections.

Recommended Tee Length and Flange and Web Thicknesses

To provide for stability during erection, it is recommended that the mimimum tee length be one-half the *T*-dimension of the beam to be supported. The maximum length of the tee must be compatible with the *T*-dimension of an uncoped beam and the remaining web depth, exclusive of fillets, of a coped beam. Note that the tee may encroach upon the fillet(s) as given in Figure 10–3.

To provide for flexibility, the tee selected should meet the ductility checks illustrated in Part 9. The flange thickness of tees used in simple shear connections should be held to a minimum to permit the flexure necessary to accommodate the end rotation of the beam, unless the tee connection is proportioned to meet the geometric requirements for single-plate connections.

Shop and Field Practices

When framing to a column flange, provision must be made for possible mill variation in the depth of the columns. If the tee is shop-attached to the column flange, play in the open holes usually furnishes the necessary adjustment to compensate for the mill variation. This approach offers the advantage of side erection of the beam. Alternatively, if the tee is shop-attached to the supported beam web, the beam length could be shortened to provide for mill overrun and shims could be furnished at the appropriate intervals to fill the resulting gaps or to provide for mill underrun.

When a single vertical row of bolts is used in a tee stem, a 4-in. or 5-in. stem is required to accommodate the end distance of the supported beam and possible overrun/underrun in beam length. A double vertical row of bolts will require a 7-in. or 8-in. tee stem. There is no maximum limit on L_{eh} for the tee stem.

SHEAR SPLICES

Shear splices are usually made with a single plate, as shown in Figure 10–16a, or two plates, as shown in Figures 10–16b and 10–16c. Although the rotational flexibility required at a shear splice is usually much less than that required at the end of a simple-span beam, when a highly flexible splice is desired, the splice utilizing four framing angles, shown in Figure 10–17, is especially useful. These shear splices may be bolted and/or welded.

The available strength of a shear splice is determined from the applicable limit-states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

Eccentricity must be considered in the design of shear splices, with the exception of all-bolted shear splices utilizing four framing angles, as illustrated in Figure 10–17. When the splice is symmetrical, as shown for the bolted splice in Figure 10–16a, each side of the splice is equally restrained regardless of the relative flexibility of the spliced members. Accordingly, as illustrated in Figure 10–18, the eccentricity of the shear to the center of gravity of either bolt group is equal to half the distance between the centroids of the bolt groups. Therefore, each bolt group can be designed for the shear, R_u or R_a, and one-half the eccentric moment, $R_u e$ or $R_a e$ (Kulak and Green, 1990). This approach is also applicable to symmetrical welded splices.

When the splice is not symmetrical, as shown in Figures 10–16b and 10–16c, one side of the splice will possess a higher degree of rigidity. For the splice shown in Figure 10–16b, the right side is more rigid because the stiffness of the weld group exceeds the stiffness of the bolt group, even if the bolts are pretensioned or slip-critical. Also, for the splice shown in Figure 10–16c, the right side is more rigid since there are two vertical rows of bolts while the left side has only one. In these cases, it is conservative to design the side with the higher rigidity for the shear, R_u or R_a, and the full eccentric moment R_ue or R_ae. The side with the lower rigidity can then be designed for the shear only. This approach is applicable regardless of the relative flexibility of the spliced members.

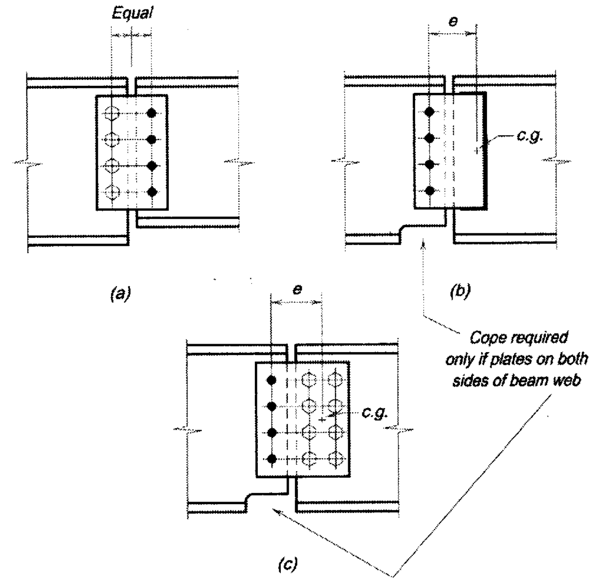


Figure 10–16. Plate-type shear splices.

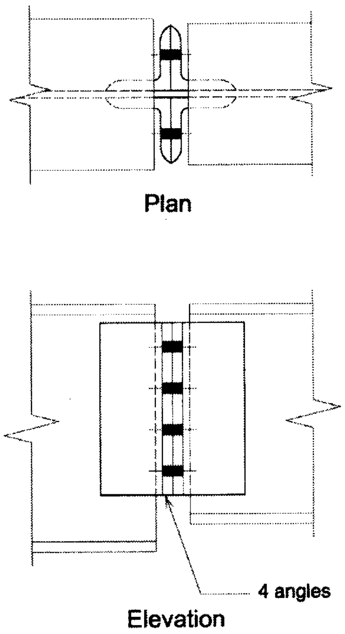


Figure 10–17. Angle-type shear splice.

Some splices, such as those that occur at expansion joints, require special attention and are beyond the scope of this Manual.

SPECIAL CONSIDERATIONS FOR SIMPLE SHEAR CONNECTIONS

Simple Shear Connections Subject to Axial Forces

When simple shear connections are subjected to axial load in addition to the shear, the important limit-states are angle leg bending and prying action. These tend to require that the angle, plate, or flange thickness increase or the gage decrease, or both, and these requirements may compromise the connection's ability to remain flexible enough to accommodate the simple beam end rotation. The shear connection ductility checks derived in Part 9 can be used to ensure that adequate ductility exists.

Simple Shear Connections at Stiffened Column-Web Locations

Stiffeners are obstacles to direct connections to the column web. Figure 10–19a illustrates a seat angle-welded to the toes of the column flanges; Figure 10–19d shows a vertical plate extended beyond the column flanges. Figures 10–19b and 10–19c offer two additional options for framing at locations of diagonal stiffeners; these should be examined carefully as they may create erection problems. Additionally, the deep cope of Figure 10–19c may significantly reduce the available strength of the beam at the end connection. Alternatively, the bottom transverse stiffener could be extended to serve as a seat plate with a bearing stiffener provided to distribute the beam reaction.

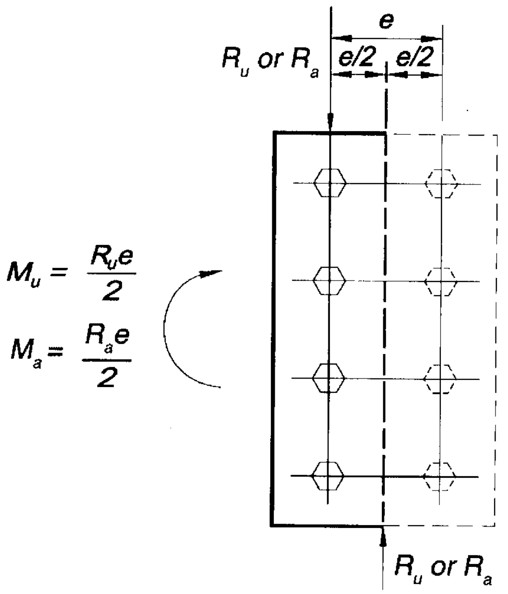


Figure 10–18. Eccentricity in a symmetrical shear splice.

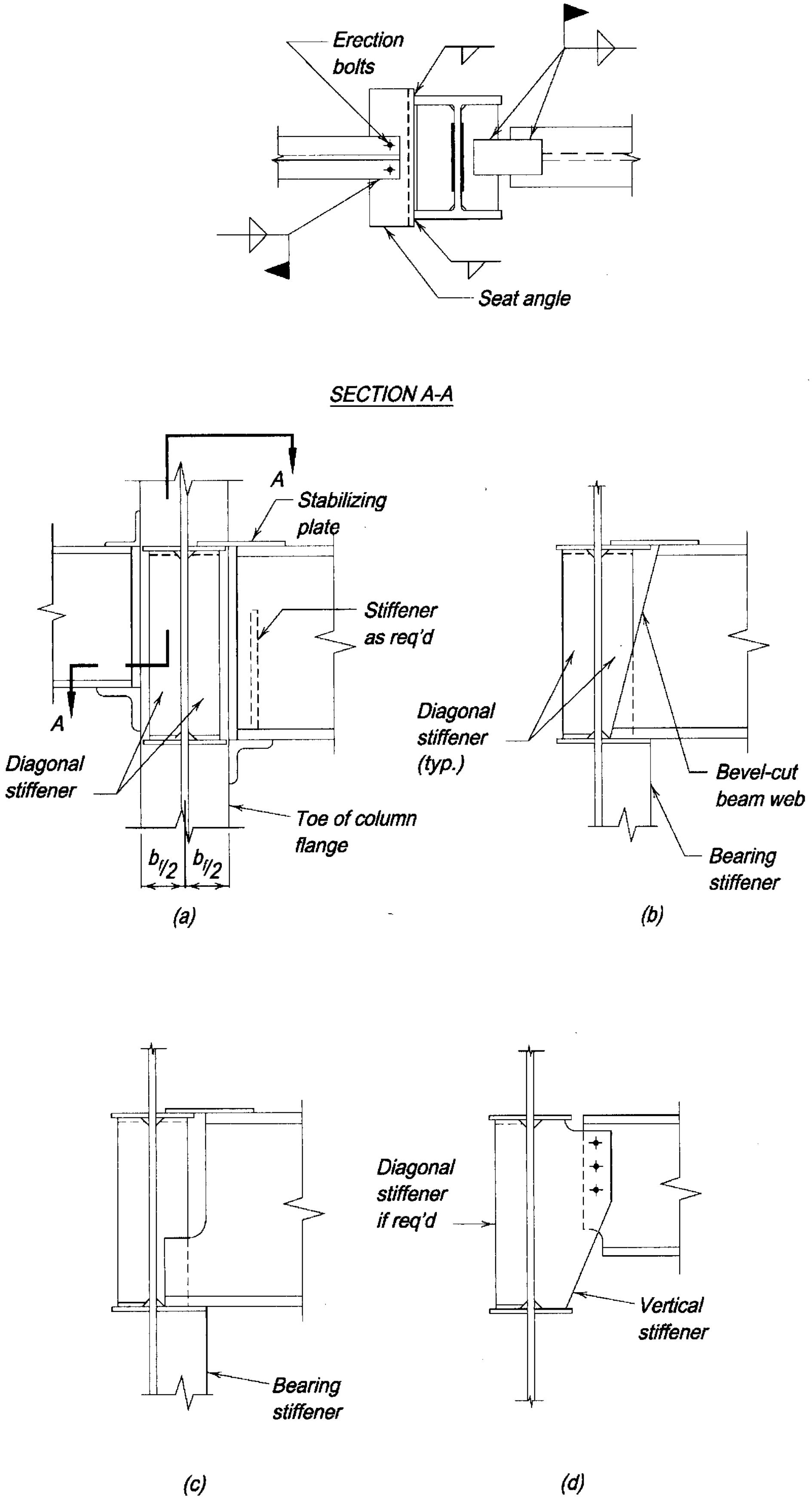


Figure 10–19. Simple shear connections at stiffened column-web locations.

Eccentric Effect of Extended Gages

Consider a simple shear connection to the web of a column that requires transverse stiffeners for two concurrent beam-to-column-flange moment connections. If it were not possible to eliminate the stiffeners by selection of a heavier column section, the field connection would have to be located clear of the column flanges, as shown in Figure 10–20, to provide for access and erectability.

The extension of the connection beyond normal gage lines results in an eccentric moment. While this eccentric moment is usually neglected in a connection framing to a column flange, the resistance of the column to weak-axis bending is typically only 20 to 50 percent of that in the strong axis. Thus the eccentric moment should be considered in this column-web connection, especially if the eccentricity, e, is large. Similarly, eccentricities larger than normal gages may also be a concern in connections to girder webs.

Column-Web Supports

There are two components contributing to the total eccentric moment: (1) the eccentricity of the beam end reaction, Re; and (2) M_{pr}, the partial restraint of the connection. To determine what eccentric moment must be considered in the design, first assume that the column is part of a braced frame for weak-axis bending, is pinned-ended with $K = 1$, and will be concentrically loaded, as illustrated in Figure 10–21. The beam is loaded before the column and will deflect under load as shown in Figure 10–22. Because of the partial restraint of the connection, a couple, M_{pr}, develops between the beam and column and adds to the eccentric couple, Re. Thus, $M_{con} = Re + M_{pr}$.

As the loading of the column begins, the assembly will deflect further in the same direction under load, as indicated in Figure 10–23, until the column load reaches some magnitude P_{sbr} when the rotation of the column will equal the simply supported beam end rotation. At this load, the rotation of the column negates M_{pr} since it also relieves the partial restraint effect of the connection, and $M_{con} = Re$. As the column load is increased above P_{sbr}, the column rotation exceeds the simply supported beam end rotation and a moment M'_{pr} results such that $M_{con} = Re - M'_{pr}$.

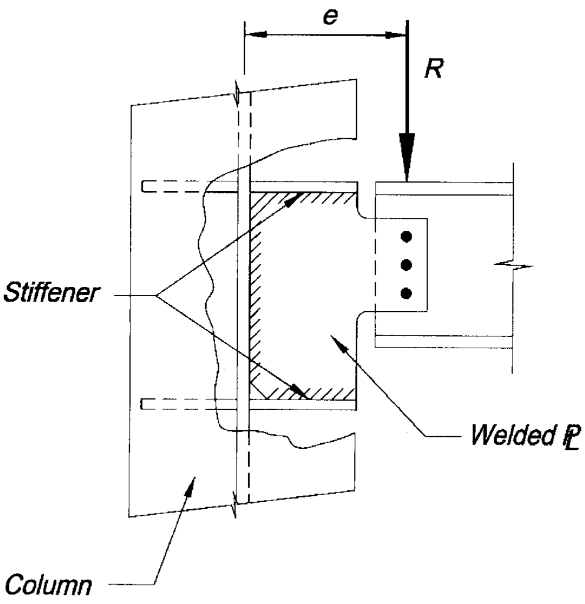


Figure 10–20. Eccentric effect of extended gages.

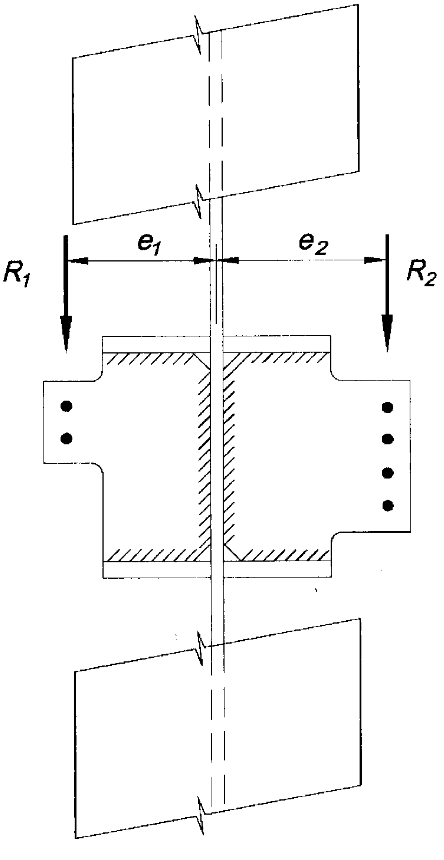


Figure 10–21. Column subject to dual eccentric moments.

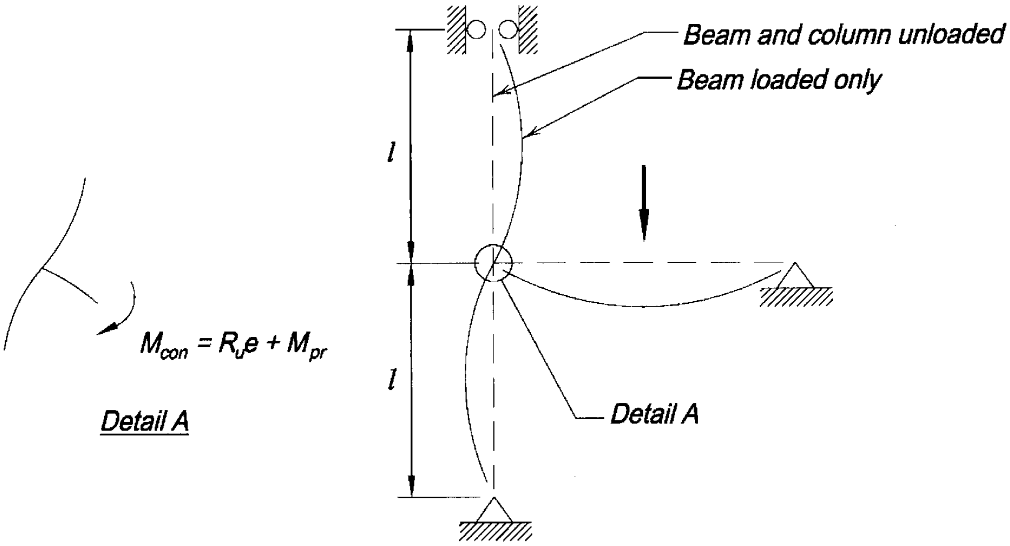


Figure 10–22. Illustration of beam, column, and connection behavior under loading of beam only.

Note that the partial restraint of the connection now actually stabilizes the column and reduces its effective length factor K below the originally assumed value of 1. Thus, since M'_{pr} must be greater than zero, it must also be true that $Re > M_{con}$. It is therefore conservative to design the connection for the shear, R and the eccentric moment, Re.

The welds connecting the plate to the supporting column web should be designed to resist the full shear, R only; the top and bottom plate-to-stiffener welds have minimal strength normal to their length, are not assumed to carry any calculated force, and may be of minimum size in accordance with AISC Specification Section J2.

If simple shear connections frame to both sides of the column web, as illustrated in Figure 10–21, each connection should be designed for its respective shear, R_1, and R_2 and the eccentric moment $|R_2e_2 - R_1e_1|$ may be apportioned between the two simple shear connections as the designer sees fit; the total eccentric moment may be assumed to act on the larger connection, the moment may be divided proportionally among the connections according to the polar moments of inertia of the bolt groups (relative stiffness), or the moment may be divided proportionally between the connections according to the section moduli of the bolt groups (relative moment strength). If provision is made for ductility and stability, it follows from the lower bound theorem of limit states analysis that the distribution which yields the greatest strength is closest to the true strength. Note that the possibility exists that one of the beams may be devoid of live load at the same time that the opposite beam is fully loaded. This condition must be considered by the designer when apportioning the moment.

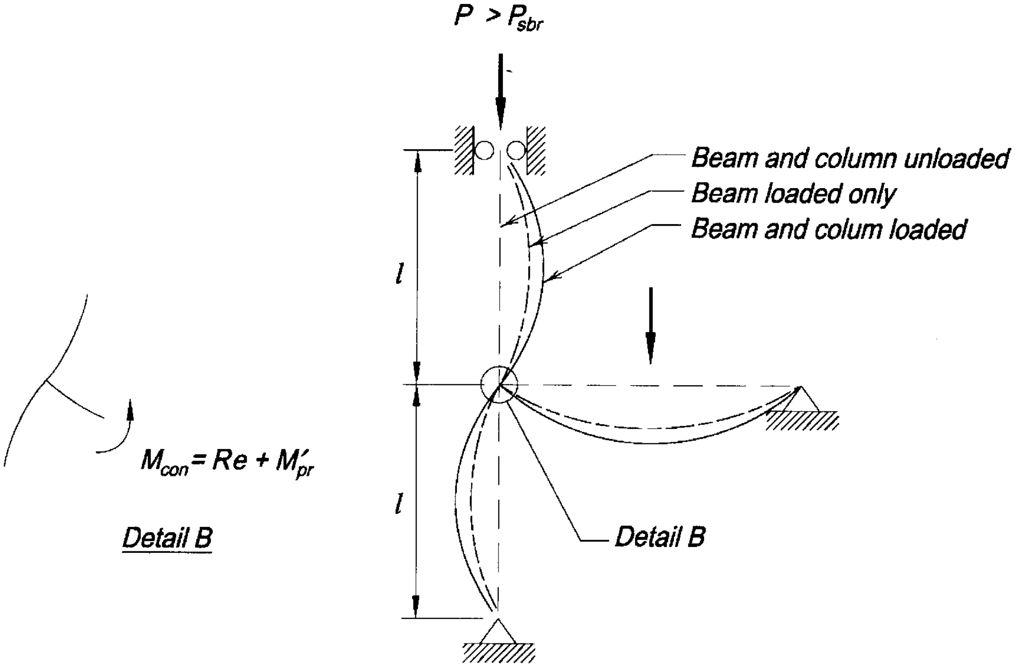


Figure 10–23. Illustration of beam, column, and connection behavior under loading of beam and column.

Girder-Web Supports

The girder-web support of Figure 10–24 usually provides only minimal torsional stiffness or strength. When larger-than-normal gages are used, the end rotation of the supported beam will usually be accommodated through rotation of the girder support. It follows that the bolt group should be designed to resist both the shear, R, and the eccentric moment, Re. The beam end reaction will then be carried through to the center of the supporting girder web.

The welds connecting the plate to the supporting girder web should be designed to resist the shear, R, only; the top and bottom plate-to-girder-flange welds have minimal strength normal to their length, are not assumed to carry any calculated force, and may be of minimum size in accordance with AISC Specification Section J2.

Similarly, for the girder illustrated in Figure 10–25 supporting two eccentric reactions, each connection should be designed for its respective shear R_1 and R_2, and the eccentric moment, $|R_2e_2 - R_1e_1|$, may be apportioned between the two simple shear connections as the designer sees fit.

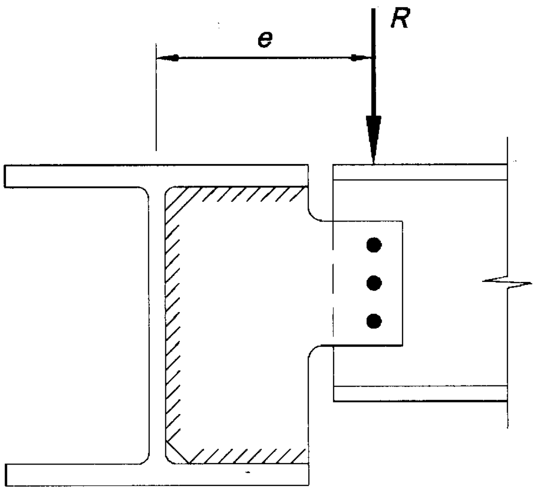


Figure 10–24. Eccentric moment on girder-web support.

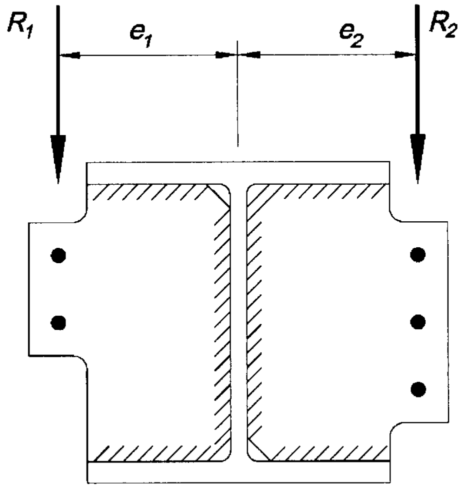


Figure 10–25. Girder-web support subject to dual eccentric moments.

Alternative Treatment of Eccentric Moment

In the foregoing treatment of eccentric moments with column- and girder-web supports, it is possible to design the support (instead of the connection) for the eccentric moment Re. Additionally, when metal deck is used with puddle welds or self-tapping screws, the metal deck tends to reduce relative movement between the two members and thus will tend to carry all or some of the eccentric moment. In these cases, the connection may be designed for the shear, R, only or the shear and a reduced eccentric moment.

Double Connections

When beams frame opposite each other and are welded to the web of the supporting girder or column, there are usually no dimensional constraints imposed on one connection by the presence of the other connection unless erection bolts are common to each connection. When the connections are bolted to the web of the supporting column or girder, however, the close proximity of the connections requires that some or all fasteners be common to both connections. This is known as a double connection. See also the discussion under Erectability Considerations.

Supported Beams of Different Nominal Depths

When beams of different nominal depths frame into a double connection, care must be taken to avoid interference from the bottom flange of the shallower beam with the entering and tightening clearances for the bolts of the connection for the deeper beam. Access to the bolts that will support the deeper beam may be provided by coping or blocking the bottom flange of the shallower beam. Alternatively, stagger may be used to favorably position the bolts around the bottom flange of the shallower beam.

Supported Beams Offset Laterally

Frequently, beams do not frame exactly opposite each other, but are offset slightly, as illustrated in Figure 10–26. Several connection configurations are possible, depending on the offset dimension.

If the offset were equal to the gage on the support, the connection could be designed with all bolts on the same gage lines, as shown in Figure 10–26b, and the angles arranged, as shown in Figure 10–26d. If the offset were less than the gage on the support, staggering the bolts, as shown in Figure 10–26c, would reduce the required gage and the angles could be arranged, as shown in Figure 10–26c. In any case, each bolt transmits an equal share of its beam reaction(s) to the supporting member, with the bolts that are loaded in double shear ultimately carrying twice as much force as those loaded in single shear. Once the geometry of the connection has been determined, the distribution of the forces is patterned after that in the design of a typical connection. For normal gages, eccentricity may be ignored in this type of connection.

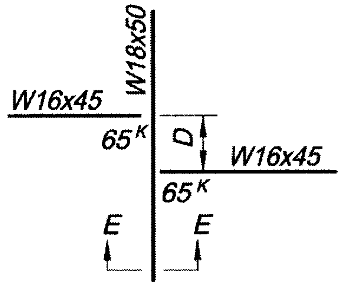
Beams Offset From Column Centerline

Framing to the Column Flange from the Strong Axis

As illustrated in Figure 10–27, beam-to-column-flange connections offset from the column centerline may be supported on a typical welded seat, stiffened or unstiffened, provided the welds for the seat can be spaced approximately equally on either side of the beam centerline.

Two such seats offset from the W12×65 column centerline by $2^1/4$ in. and $3^1/2$ in. are shown in Figures 10–27a and 10–27b, respectively. While not shown, top angles should be used with this connection.

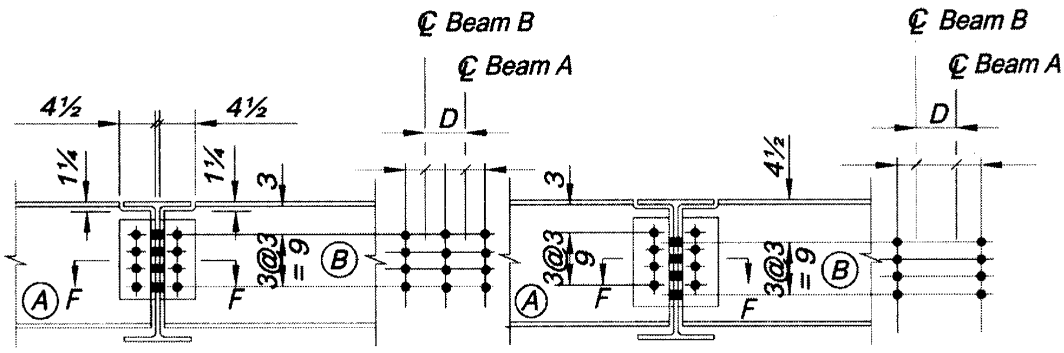
Since the entire seat fits within the flange width of the column, the connection of Figure 10–27a is readily selected from the design aids presented previously. However, the larger



PART PLAN

(Beam flush top)

(a)



SECTION E - E

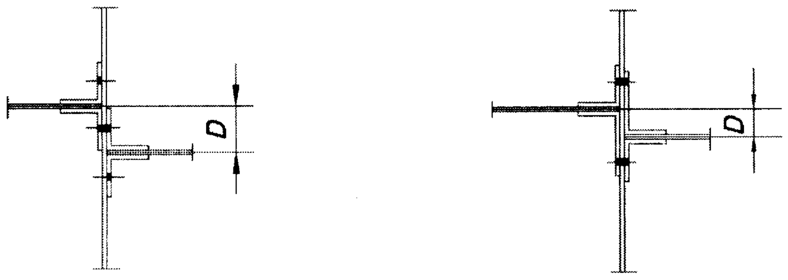
Bolts on same gage

(b)

SECTION E - E

Bolts staggered

(c)



SECTION F - F

Bolts on same gage

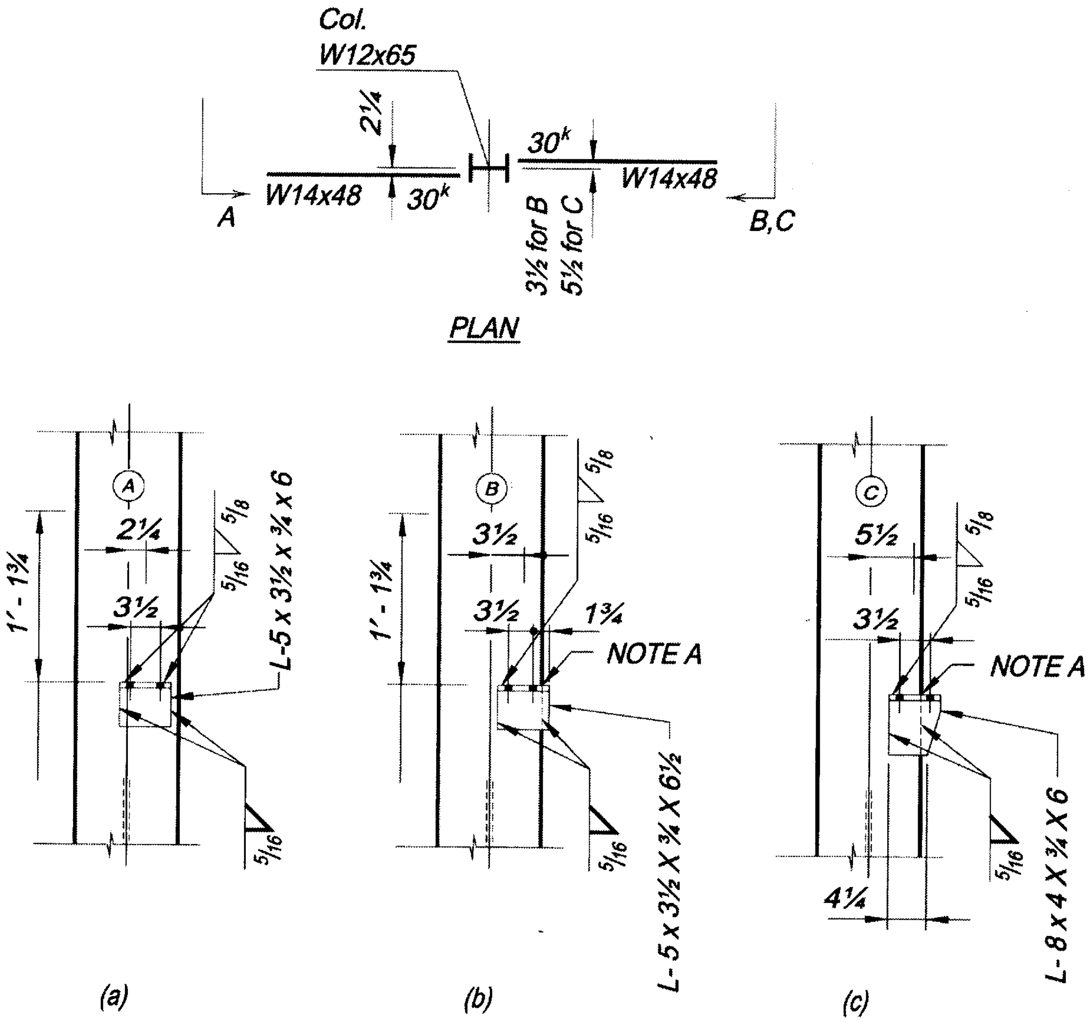
(d)

SECTION F - F

Bolts staggered

(e)

Figure 10–26. Offset beams connected to girder.



NOTE A

End return is omitted because the AWS Code does not permit weld returns to be carried around the corner formed by the column flange toe and seat angle heel.

NOTE B

Beam and top angle not shown for clarity.

Figure 10–27. Offset beams connected column flanges.

beam offsets in Figures 10–27b and 10–27c require that one of the welds be made along the edge of the column flange against the back side of the seat angle. Note that the end return is omitted because weld returns should not be carried around such a corner.

For the beam offset of $5^1/_2$ in. shown in Figure 10–27c, the seat angle overhangs the edge of the beam and the horizontal distance between the vertical welds is reduced to $3^1/_2$ in.; the center of gravity of the weld group is located $1^1/_4$ in. to the left of the beam centerline. The force on each weld may be determined by statics. In this case, the larger force is in the right-hand weld and may be determined by summing moments about the lefthand weld. Once the larger force has been determined, the seat should conservatively be designed to carry twice the force in the more highly loaded weld.

Framing to the Column Flange from the Weak Axis

Spandrel beams X and Y in the partial plan shown in Figure 10–28 are offset $4^1/_8$ in. from the centerline of column C1, permitting the beam web to be connected directly to the column flange. At column B2, spandrel beam X is offset five inches and requires a $^7/_8$-in. filler between the beam web and the column flange. Beams X and Y are both plain-punched beams, with flange cuts on one side, as noted in Figure 10–28a, Section F-F.

In establishing gages, the requirements of other connections to the column at adjacent locations must be considered. While the workable flange gage is $3^1/_2$ in. for the W8×28 columns supporting the spandrel beams, for beams Z, the combination of a 4-in. column gage and $1^1/_2$-in. stagger of fasteners is used to provide entering and tightening clearance for the field bolts and sufficient edge distance on the column flange, as illustrated in Figure 10–28b. The 4-in. column gage also permits a $1^1/_2$-in. edge distance at the ends of the spandrel beams, which will accommodate the normal length tolerance of $\pm^1/_4$ in. as specified in "Standard Mill Practice" in Part 1.

The spandrel beams are shown with the notation "Cut and Grind Flush FS" in Sections E-E and F-F. This cut permits the beam web to lie flush against the column flange. The uncut flange on the near side of the spandrel beam contributes to the stiffness of the connection. The $2^1/_2 \times ^7/_8$-in. filler is required between the spandrel beam web and the flange of the column B2 because of the $^7/_8$-in. offset. Since the filler in Section E-E, Figure 10–28a, is thicker than $^3/_4$ in., it must be fully developed.

In the part plan in Figure 10–29a, the W16×40 beam is offset $6^1/_4$ in. from the centerline of column D1. This prevents the web of the W16×40 from being placed flush against the side of the column flange. A plate and filler are used to connect the beam to the column flange, as shown in Figure 10–29b. Such a connection is eccentric and one group of fasteners must be designed for the eccentricity. Lack of space on the inner flange face of the column requires development of the moment induced by the eccentricity in the beam web fasteners.

To minimize the number of field fasteners, the plate in this case is shop-bolted to the beam and field-bolted to the column. A careful check must be made to ensure that the beam can be erected without interference from fittings on the column web. Some fabricators would elect to shop-attach the plate to the column to eliminate possible interference and permit use of plain-punched beams. Additionally, if the column were a heavy section, the fabricator may elect to shop-weld the plate to the column to avoid drilling the thick flanges. The welding of this plate to the column creates a much stiffer connection and the design should be modified to recognize the increased rigidity.

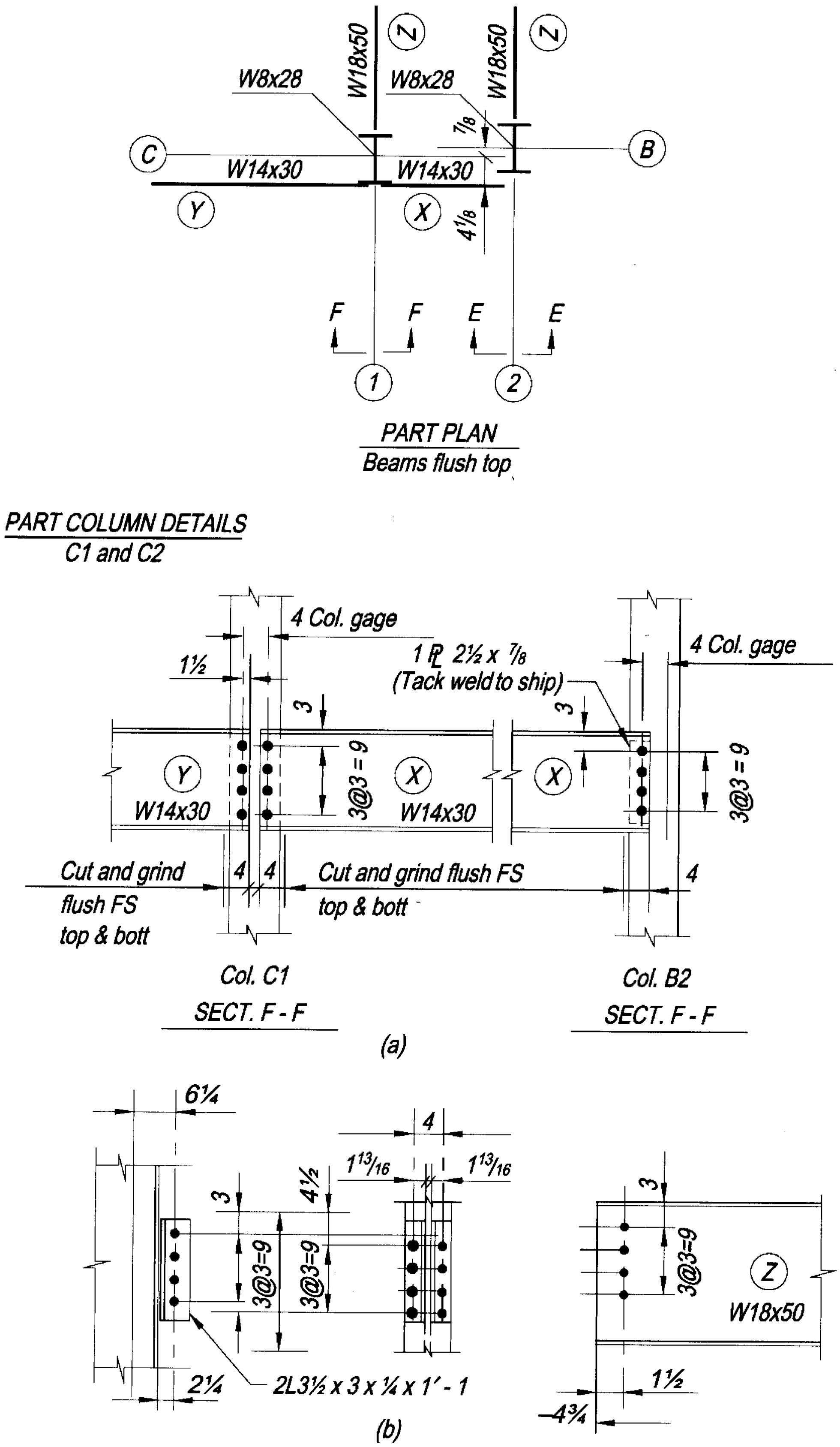


Figure 10–28. Offset beams connected to column.

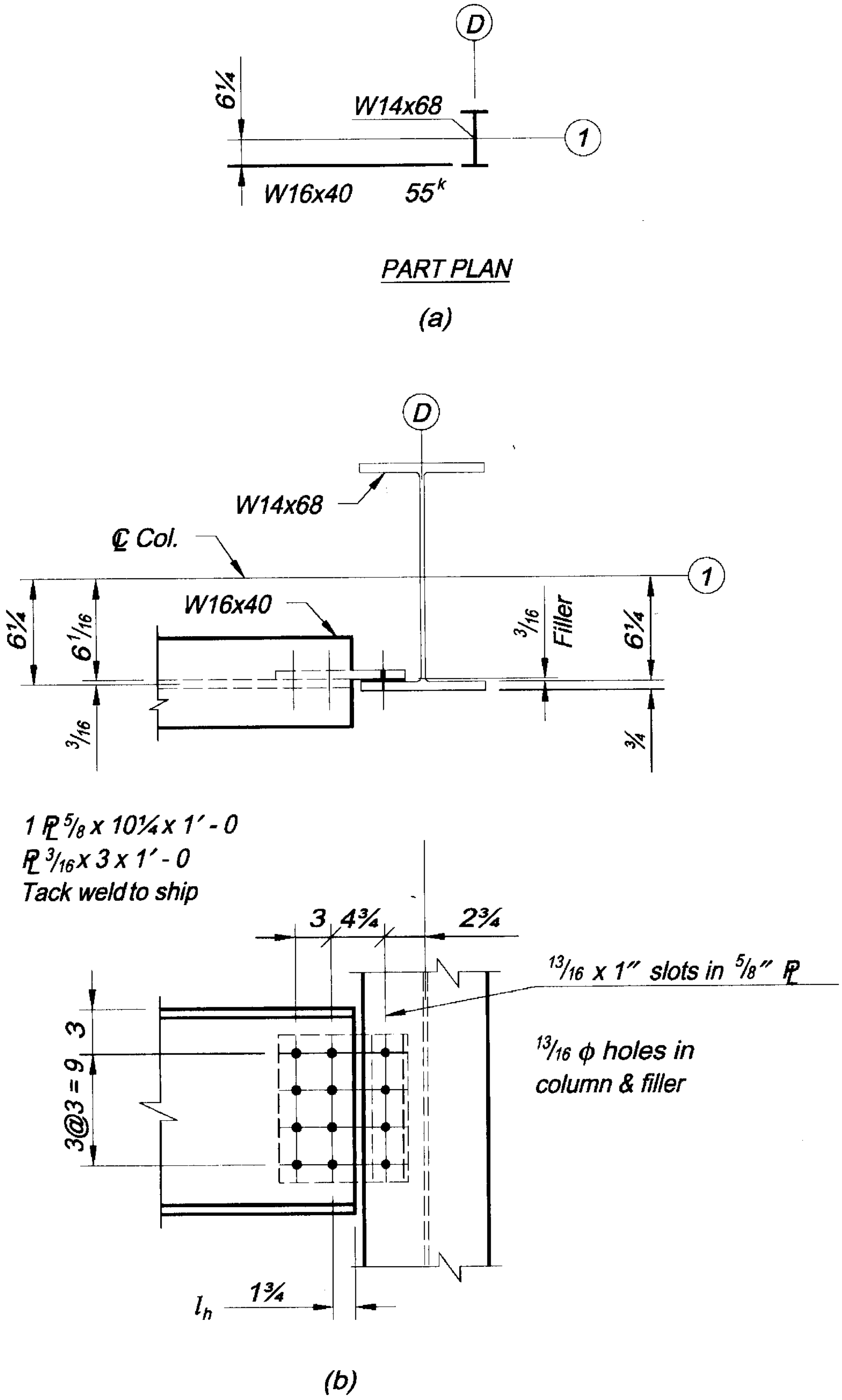


Figure 10–29. Offset beam connected to column.

If the centerline of the W16 were offset 6^1/16 in. from line 1, it would be possible to cope or cut the flanges flush top and bottom and frame the web directly to the column flange with details similar to those shown in Figure 10–29. This type of framing also provides a connection with more rigidity than normally contemplated in simple construction. A coped connection of this type would create a bending moment at the root of the cope that might require reinforcement of the beam web.

One method frequently adopted to avoid moment transfer to the column because of beam connection rigidity is to use slotted holes and a bearing connection to provide some flexibility. The slotted holes would be provided in the connection plate only and would be in the field connection only. These slotted connections also would accommodate fabrication and erection tolerances.

The type of connection detailed in Figure 10–29 is similar to a coped beam and should be checked for buckling, as illustrated in Part 9. The following differences are apparent and should be recognized in the analysis:

1. The effective length of equivalent "cope" is longer by the amount of end distance to the first bolt gage line.
2. There is an inherent eccentricity due to the beam web and plate thickness. The ordinary web and plate thicknesses normally will not require an analysis for this condition, since the inelastic rotation allowed by the AISC Specification will relieve this secondary moment effect. Two plates may sometimes be required to counter this eccentricity when dimensions are significant.
3. The connection plate can be made of sufficient thickness as required for bending or buckling stresses with a minimum thickness of ^{3}/8 in.

Framing to the Column Web

If the offset of the beam from the centerline of the column web is small enough that the connection may still be centered on or under the supported beam, no special considerations need be made. However, when the offset of the beam is too large to permit the centering of the connection under the beam, as in Figure 10–30, it may be necessary to consider the effect of eccentricity in the fastener group.

The offset of the beam in Figure 10–30 requires that the top and bottom flanges be blocked to provide erection clearance at the column flange. Since only half of each flange, then, remains in which to punch holes, a 6-in. outstanding leg is used for both the seat and top angles of these connections; this permits the use of two field bolts to each of the seat and top angles, which are required by OSHA.

Connections for Raised Beams

When raised beams are connected to column flanges or webs, there is usually no special consideration required. However, when the support is a girder, the differing tops of steel may preclude the use of typical connections. Figure 10–31 shows several typical details commonly used for such cases in bolted construction. Figure 10–32 shows several typical details commonly used in welded construction.

In Figure 10–31a, since the top of the W12×35 is located somewhat less than 12 in. above the top of the W18 supporting beam, a double-angle connection is used. This connection would be designed for the beam reaction and the shop bolts would be governed by double shear or bearing, just as if they were located in a vertical position. However, the field bolts are not required to carry any calculated force under gravity loading.

The maximum permissible distance *m* depends on the beam reaction, since the web remaining after the bottom cope must provide sufficient area to resist the vertical shear as well as the bending moment which would be critical at the end of the cope. The beam can be reinforced by extending the angles beyond the cope and adding additional shop bolts for development. The angle size and/or thickness can be increased to gain shear area or section modulus, if required. The effect of any eccentricity would be a matter of judgment, but could be neglected for small dimensions.

When this connection is used for flexure or for dynamic or cyclical loading, the web is subjected to high stress concentrations at the end of the cope, and it is good practice to extend the angles, as shown in Figure 10–31a by the dashed lines, to add at least two additional web fasteners.

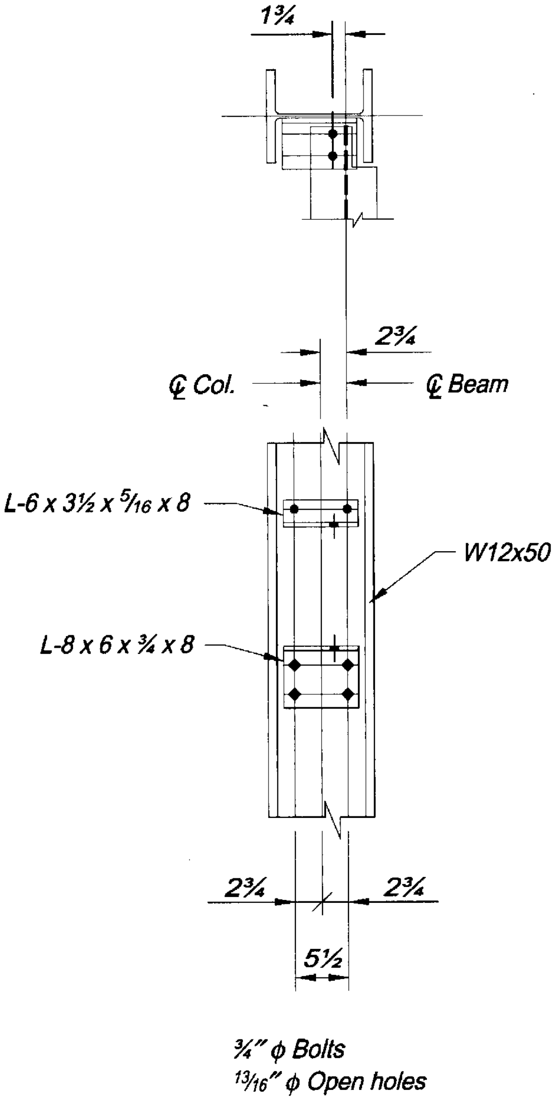


Figure 10–30. Offset beam connected to column web.

Figure 10–31b covers the case where the bottom flange of the W12×35 is located a few inches above the top of the W18. The beam bears directly upon fillers and is connected to the W18 by four field bolts which are not required to transmit a calculated gravity load. If the distance *m* exceeds the thickest plate which can be punched, two or more plates may be used. Even though the fillers in this case need only be $6^1/2$-in. square, the amount of material required increases rapidly as *m* increases. If *m* exceeds 2 or 3 in., another type of detail may be more economical.

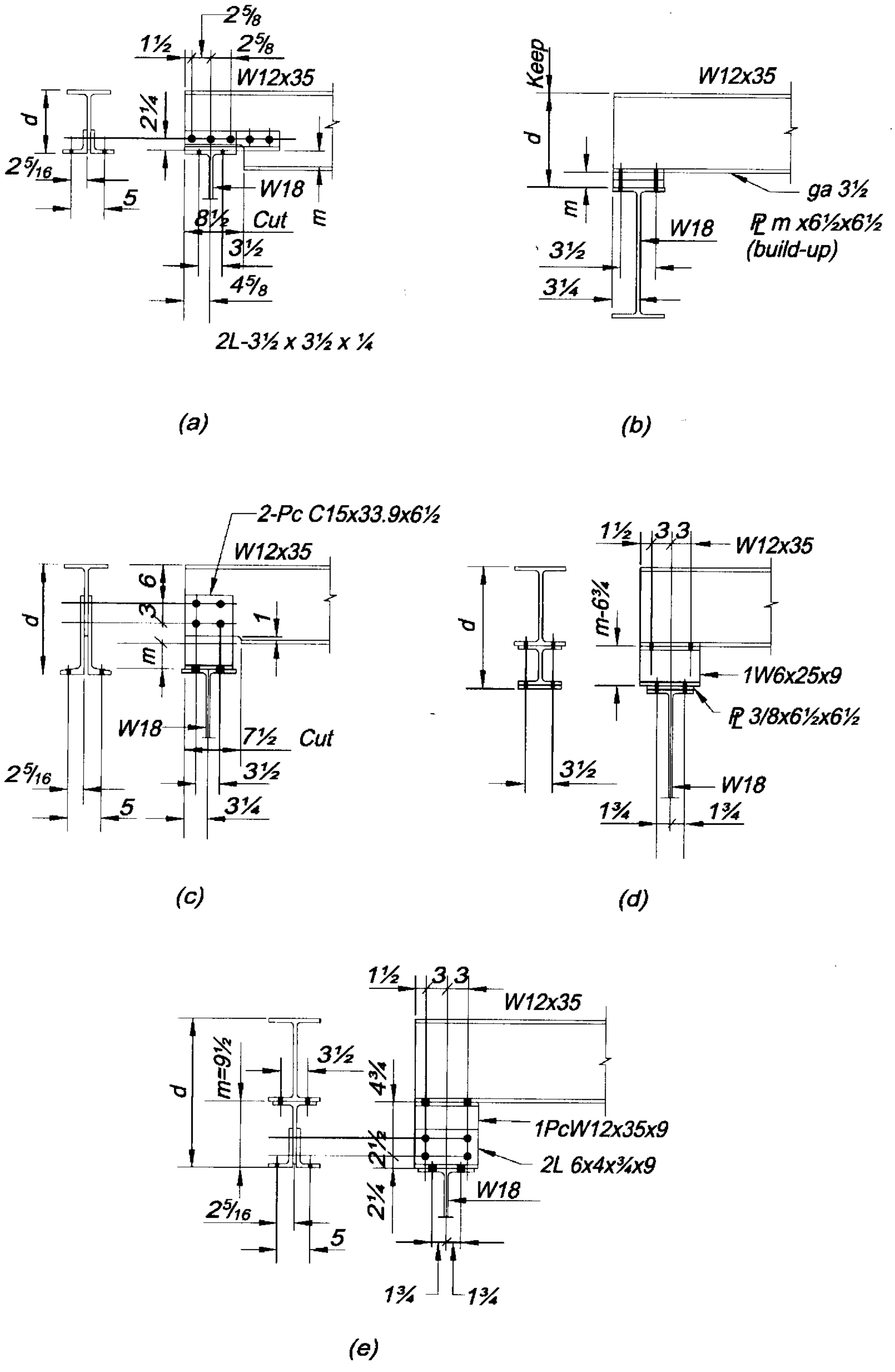
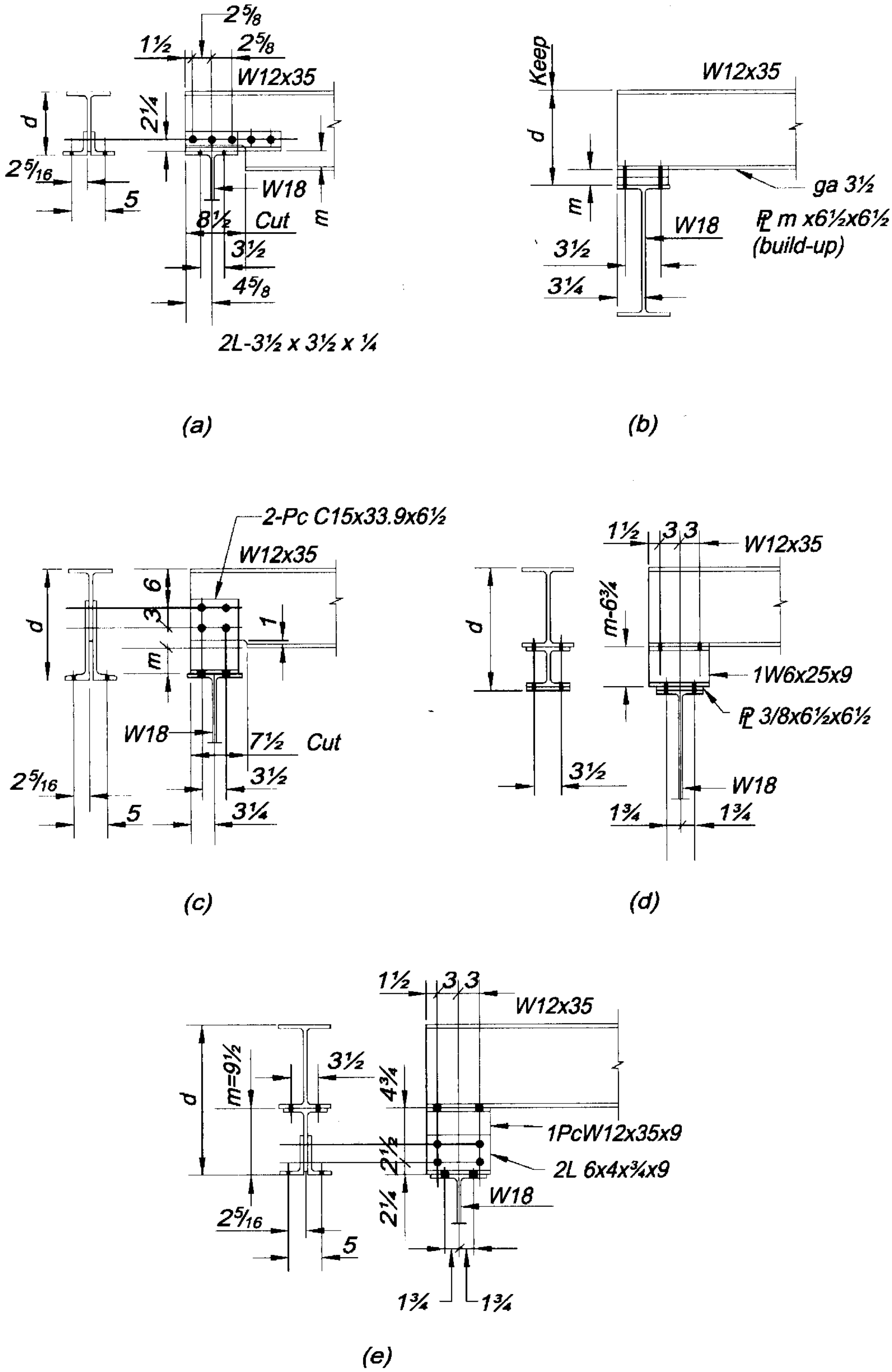


Figure 10–31. Bolted raised-beam connections.

The detail shown in Figure 10–31c is used frequently when *m* is up to 6 or 7 in. The load on the shop bolts in this case is no greater than that in Figure 10–31a. However, to provide more lateral stiffness, the fittings are cut from a 15-in. channel and are detailed to overlap the beam web sufficiently to permit four shop bolts on two gage lines.

A stool or pedestal, cut from a rolled shape, can be used with or without fillers to provide for the necessary *m* distance, as in Figure 10–31d. A pair of connection angles and a tee will also serve a similar purpose, as shown in Figure 10–31e. To provide adequate strength to



(a) (b)

(c) (d)

(e)

Figure 10–32. Welded raised-beam connections.

carry the beam end reaction and to provide lateral stiffness, the web thickness of the pedestal in each of these cases should be at least as thick as the member being supported.

In Figure 10–32a, welded framing angles are substituted for the bolted angles of Figure 10–32a. In Figure 10–32b, a single horizontal plate is shown replacing the pair of framing angles; this results in a savings in material and the amount of shop-welding. In this case, particular care must be taken in cutting the beam web and positioning the plate at right angles to the beam web. For this reason, if only a few connections of this type are to be made, some fabricators prefer to use the angles, as in Figure 10–32a. If sufficient duplication were available to warrant making a simple jig to position the plate during welding, the solution of Figure 10–32b may be economical.

Figure 10–32c shows a tee centered on the beam web and welded to the bottom flange of the beam. The tee stem thickness should not be less than the beam web thickness. The welded solutions shown in Figures 10–32d and 10–32e are capable of providing good lateral stiffness. The latter two types also permit end rotation as the beam deflects under load. However, if the *m* distance exceeds 3 or 4 in., it is advisable to shop-weld a triangular bracket plate at one end of the beam, as indicated by the dashed lines, to prevent the beam from deflecting along its longitudinal axis.

Other equally satisfactory details may be devised to meet the needs of connections for raised beams. They will vary depending on the size of the supported beam and the distance *m*. When using this type of connection where the load is transmitted through bearing, the provisions of AISC Specification Sections J10.2 and J10.3 must be satisfied for both the supported and supporting members. For the detail of Figure 10–32b, since the rolled fillet has been removed by the cut, the value of *k* would be taken as the thickness of the plate plus the fillet weld size.

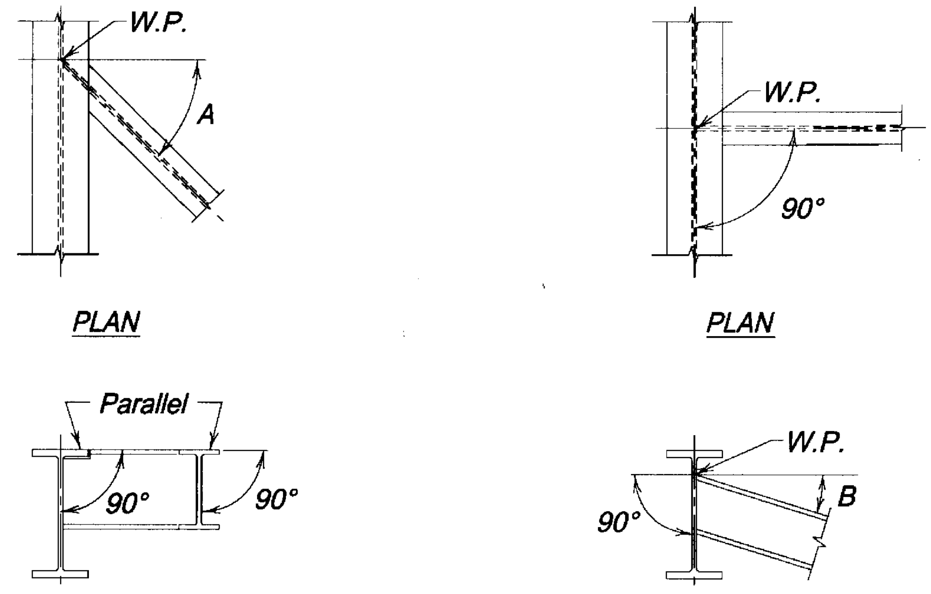
AISC Specification Appendix 6 requires stability and restraint against rotation about the beam's longitudinal axis. This provision is most easily accomplished with a floor on top of the supported beam. In the absence of a floor, the top flange may be supported by a strut or bracket attached to the supporting member. When the beam is encased in a wall, this stability may also be provided with wall anchors; refer to "Wall Anchors" in Part 15.

This discussion has considered that the field bolts which attach the beam to the pedestal or support beam are subject to no calculated load. It is important, however, to recognize that when the beam deflects about its neutral axis, a tensile force can be exerted on the outside bolts. The intensity of this tensile force is a function of the dimension *d,* indicated in Figure 10–31, the span length of the supported member, and the beam stiffness. If these forces are large, high-strength bolts should be used and the connection analyzed for the effects of prying action.

Raised-beam connections such as these are used frequently as equipment or machinery supports where it is important to maintain a true and level surface or elevation. When this tolerance becomes important, the dimension *d* should be noted "keep" to advise the fabricator of this importance, as shown in Figure 10–31b. Since the supporting beam is subject to certain camber/deflection tolerances, it also may be appropriate to furnish shim packs between the connection and the supporting member.

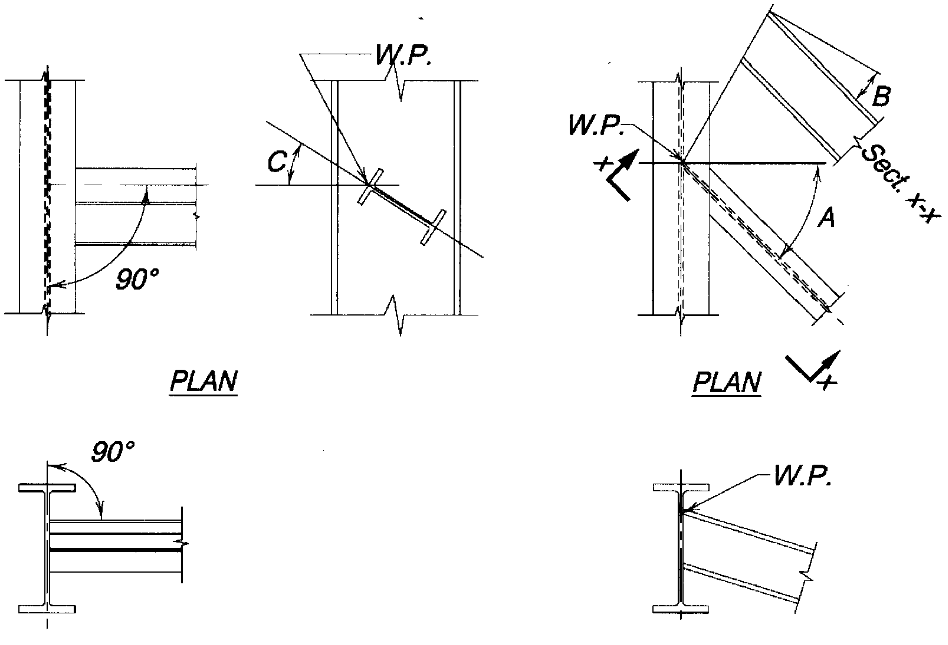
Non-Rectangular Simple Shear Connections

It is often necessary to design connections for beams that do not frame into a support orthogonally. Such a beam may be inclined with respect to the supporting member in various directions. Depending upon the relative angular position which a beam assumes, the connection may be classified among three categories: skewed, sloped, or canted. These conditions are illustrated in Figure 10–33 for beam-to-girder web connections; the same descriptions apply to beam-to-column-flange and web connections. Additionally, beams may be oriented in a combination of any or all of these conditions. For any condition of skewed, sloped, or canted framing, the single-plate connection is generally the simplest and most economical of those illustrated in this text.



(a) Skewed beam

(b) Sloped beam



(c) Canted beam

(d) Skewed and sloped beam

Figure 10–33. Non-rectangular connections.

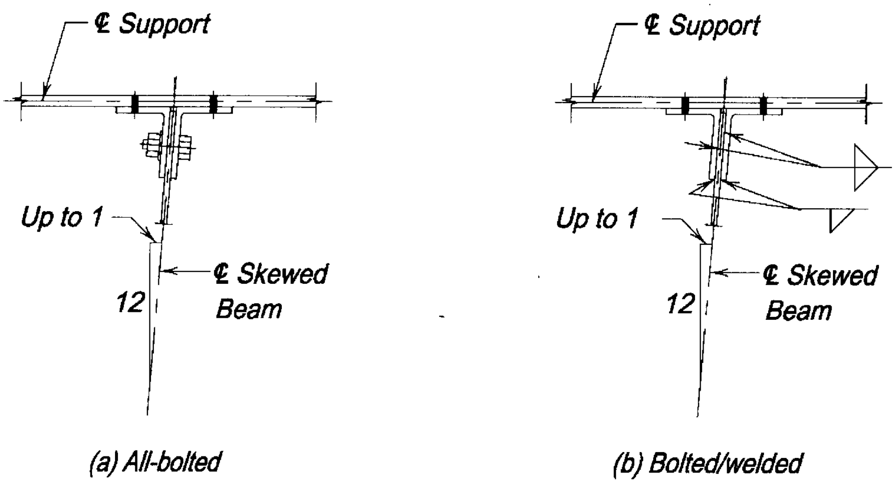
Skewed Connections

A beam is said to be skewed when its flanges lie in a plane perpendicular to the plane of the face of the supporting member, but its web inclined to the face of the supporting member. The angle of skew A appears in Figure 10–33a and represents the horizontal bevel to which the fittings must be bent or set, or the direction of gage lines on a seated connection.

When the skew angle is less than 5° (1-in-12 slope), a pair of double angles can be bent inward or outward to make the connection, as shown in Figure 10–34. While bent angle sections are usually drawn as bending in a straight line from the heel, rolled angles will tend to bend about the root of the fillet (dimension *k* in Manual Part 1). This produces a significant jog in the leg alignment, which is magnified by the amount of bend. Above this angle of skew, it becomes impractical to bend rolled angles.

For skews approximately greater than 5° (1-in-12 slope), a pair of bent plates, shown in Figure 10–35, may be a more practical solution. Bent plates are not subject to the deformation problem described for bent angles, but the radius and direction of the bend must be considered to avoid cracking during the cold-bending operation.

Bent plates exhibit better ductility when bent perpendicular to the rolling direction and are, therefore, less likely to crack. Whenever possible, bent connection plates should be



(a) All-bolted (b) Bolted/welded

Figure 10–34. Skewed beam connections with bent double angles.

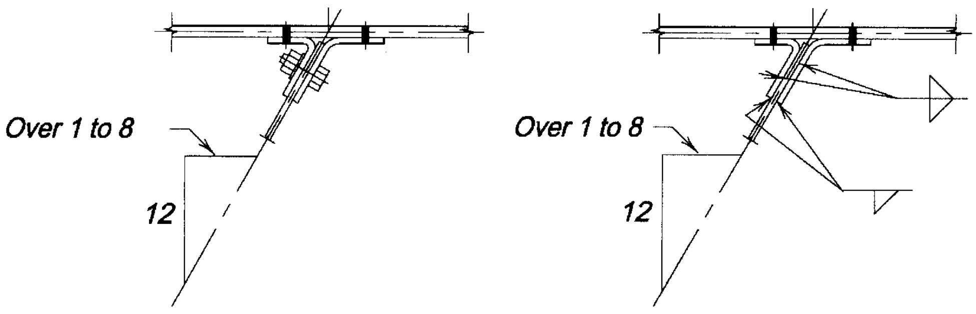


Figure 10–35. Skewed beam connections with double bent plates.

billed with the width dimension parallel to the bend line. The length of the plate is measured on its mid-thickness, without regard to the radius of the bend. While this will provide a plate that is slightly longer than necessary, this will be corrected when the bend is laid out to the proper radius prior to fabrication.

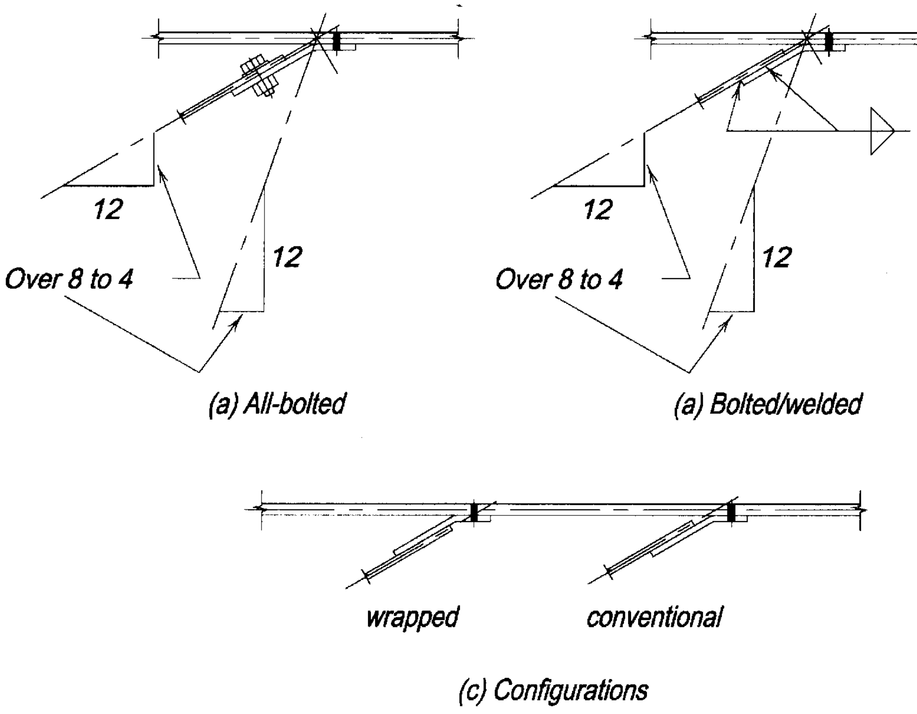
Table 10–12 gives the generally accepted minimum inside-bending radius for plate thickness, *t*, for various grades of steel. Values are for bend lines transverse to the direction of final rolling (Brockenbrough, 1998). When bend lines are parallel to the direction of final rolling, the tabular values may have to be approximately doubled. When bend lines are longer than 36 inches, all radii may have to be increased if problems in bending are encountered.

Before bending, special attention should be given to the condition of plate edges transverse to the bend lines. Flame-cut edges of hardenable steels should be machined or softened by heat treatment. Nicks should be ground out and sharp corners should be rounded.

The strength of bent angles and bent plate connections may be calculated in the same manner as for square framed beams, making due allowances for eccentricity. The load is assumed to be applied at the point where the skewed beam center line intersects the face of the supporting member.

As the angle of skew increases, entering and tightening clearances on the acutely angled side of the connection will require a larger gage on the support. If the gage were to become objectionable, a single bent plate, illustrated in Figure 10–36, may provide a better solution. Note that the single-bent plate may be of the conventional type, or a more compact connection may be developed by "wrapping" the single bent plate, as illustrated in Figure 10–36c.

In all-bolted construction, both the shop and field bolts should be designed for shear and the eccentric moment. A C-shaped weld is preferable to avoid turning the beam during shop fabrication. Single bent plates should be checked for flexural strength.



(a) All-bolted

(a) Bolted/welded

wrapped

conventional

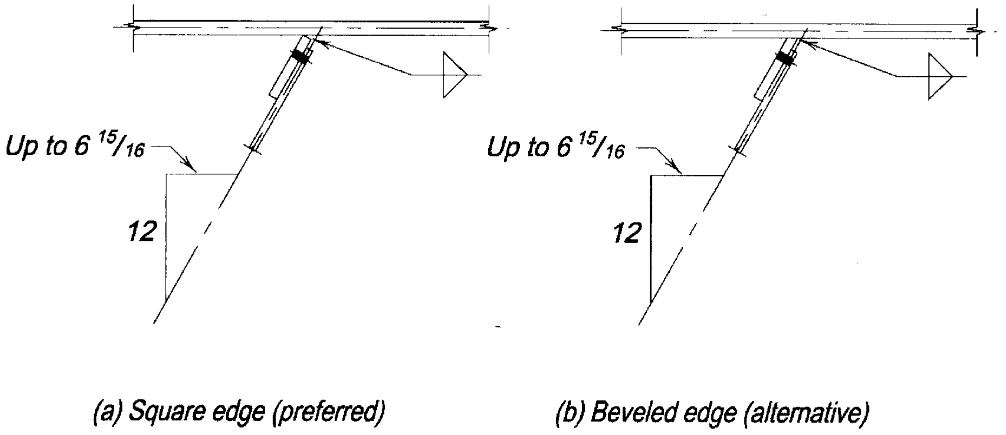
(c) Configurations

Figure 10–36. Skewed-beam connections with single-bent plates.

Table 10–13 gives clearance dimensions for bent double-angle connections and double- and single-bent plate connections, and specifies beam set-backs and gages. Since these dimensions are based on the maximum material thicknesses and fastener sizes indicated, it is suggested that in cases where many duplicate connections with less than maximum material or fasteners are required, savings can be effected if these dimensions are developed from specific bevels, beam sizes, and fitting thicknesses.

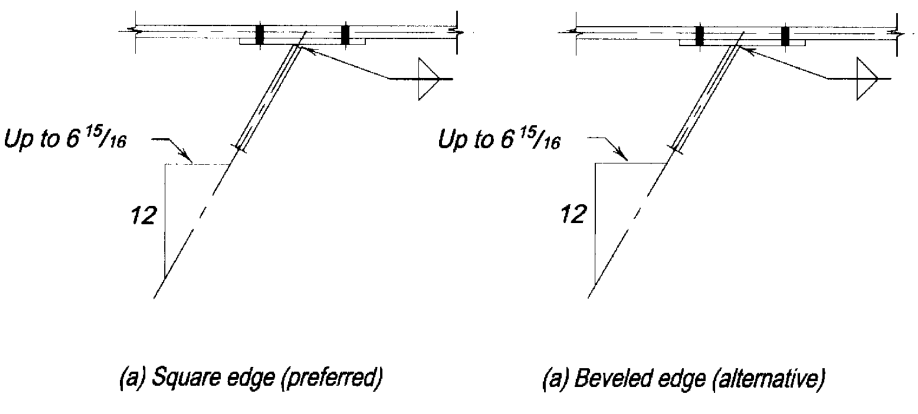
Skewed single-plate and skewed end-plate connections, shown in Figures 10–37 and 10–40, provide a simple, direct connection with a minimum of fittings and multiple punching requirements. When fillet-welded, these connections may be used for skews up to 30° (or a slope of 6⁵⁄16-in-12) provided the root opening formed does not exceed ³⁄16 in. For skew angles greater than 30°, see AWS D1.1, Section 2.2.5.2.

The maximum beam-web thickness which may be supported is a function of the maximum root opening and the angle of skew. If the thickness of the beam web were such that a larger root opening were encountered, the skewed single plate or the web connecting to the skewed end plate may be beveled, as shown in Figures 10–37b and 10–38b. Since no root opening occurs with the bevel, there is no limitation on the thickness of the beam web. However, beveling, especially of the beam web, requires careful finishing and is an expensive procedure which may outweigh its advantages.



(a) Square edge (preferred) (b) Beveled edge (alternative)

Figure 10–37. Skewed single-plate connections.



(a) Square edge (preferred) (a) Beveled edge (alternative)

Figure 10–38. Skewed shear end-plate connections.

The design of skewed end-plate connections is similar to that discussed previously in "Shear End-Plate Connections" in this Part. However, when the gage of the bolts is not centered on the beam web, this eccentric loading should be considered. The design of skewed single-plate connections is similar to that discussed previously in "Single-Plate Connections" in this Part.

Table 10–13 specifies gages and the dimension A which is added to the fillet weld size to compensate for the root opening for skewed end-plate connections. This table is based conservatively on a gap of $^1/_8$ in. For beam webs beveled to the appropriate skew, $A = 0$ and the tabulated values do not apply. Table 10–13 also provides similar information for skewed single-plate connections. Additionally, this table provides clearances and dimensions for groove-welded single-plate connections with backing bars for skews greater than 30°; refer to AWS D1.1 for prequalified welds for both types of joints.

When skewed, stiffened seated connections are used, the stiffening element should be located so as to cross the skewed beam centerline well out on the seat. This can be accomplished by shifting the stiffener to the left or right of center to support beams which skew to the left or to the right, respectively. Alternatively, it may be possible to skew the stiffening element.

Sloped Connections

A beam is said to be sloped if the plane of its web is perpendicular to the plane of the face of the supporting member, but its flanges are not perpendicular to this face. The angle of slope B is shown in Figure 10–33b and represents the vertical angle to which the fittings must be set to the web of the sloped beam, or the amount that seat and top angles must be bent.

The design of sloped connections usually can be adapted directly from the rectangular connections covered earlier in this part, with consideration of the geometry of the connection to establish the location of fittings and fasteners. Note that sloped beams often require copes to clear supporting girders, as illustrated in Figure 10–39.

Figure 10–40 shows a sloped beam with double-angle connections, welded to the beam and bolted to the support. The design of this connection is essentially similar to that for rectangular double-angle connections. Alternatively, shear end-plate, tee, single-angle, single-plate, or seated connections could be used. Selection of a particular connection type may be influenced by fabrication economy, erectability, and/or by the types of connections used elsewhere in the structure.

Sloped seated beam connections may utilize either bent angles or plates, depending on the angle of slope. Dimensioning and entering and clearance requirements for sloped seated connections are generally similar to those for skewed connections. The bent seat and top plate shown in Figure 10–41 may be used for smaller bevels.

When the angle of slope is small, it is economical to place transverse holes in the beam web on lines perpendicular to the beam flange; this requires only one stroke of a multiple punch per line. Since non-standard hole arrangements, then, usually occur in the connecting materials (which are single-punched), this requires that sufficient dimensions be provided for the connecting material to contain fasteners with adequate edges and gages, and at the same time fit the angle to the web without encroaching on the flange fillets of the beam. For the end connection of the beam, this was accomplished by using a 6-in. angle leg; a 4-in. or even a 5-in. leg would not have furnished sufficient edge distance at the extreme fastener.

As the angle of slope increases, however, bolts for the end connections cannot conveniently be lined up to permit simultaneous punching of all holes in a transverse row. In this case, the fabricator may choose to disregard beam gage lines and arrange the hole-punching so that ordinary square-framed connection material can be used throughout, as shown in Figure 10–42.

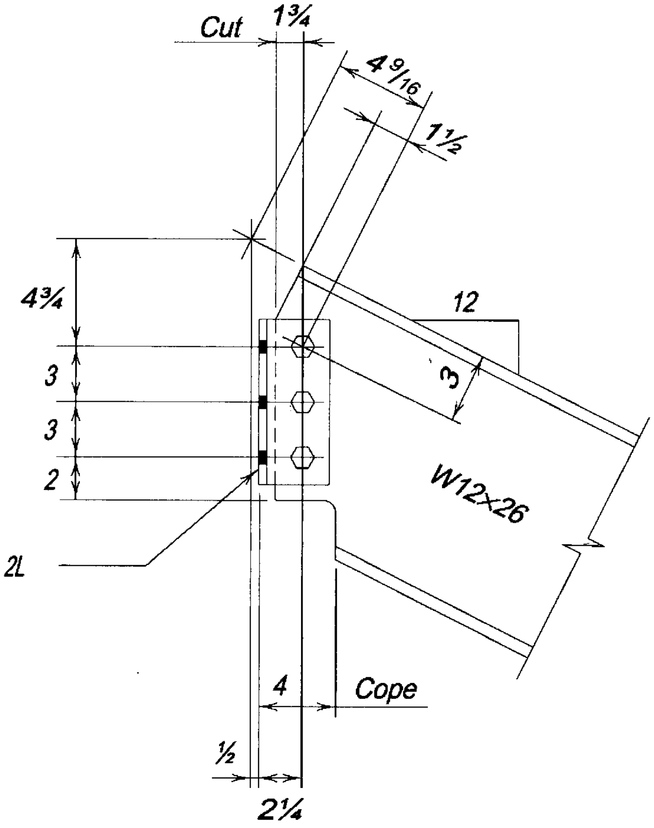


Figure 10–39. Sloped double-angle connection.

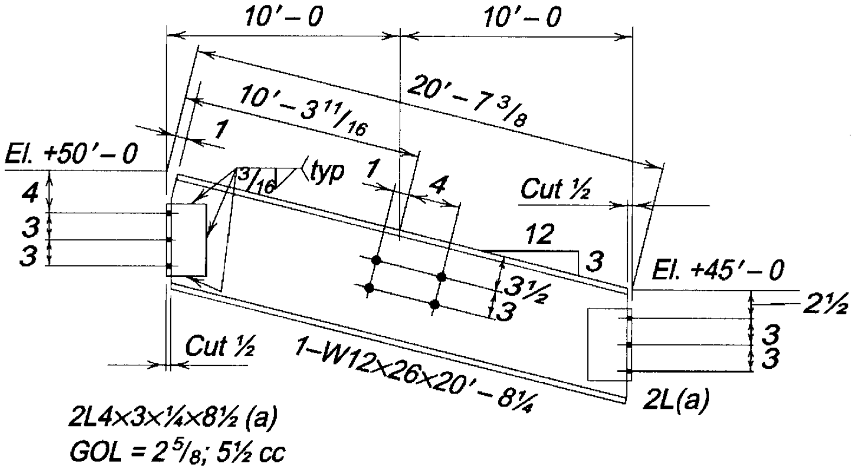


Figure 10–40. Sloped double-angle connection.

Canted Connections

A beam perpendicular to the face of a supporting member, but rotated so that its flanges are tilted with respect to those of the support, is said to be canted. The angle of cant C is shown in Figure 10–33c.

The design of canted connections usually can be adapted directly from the rectangular connections covered earlier in this part. In Figure 10–43, a double-angle connection is used.

Alternatively, shear end-plate, seated, single-angle, single-plate, and tee connections may also be used.

For channel B2, which is supported by a sloping member B1 (not shown), to match the hole pattern in supporting member B1, the holes in the connecting materials must be canted.

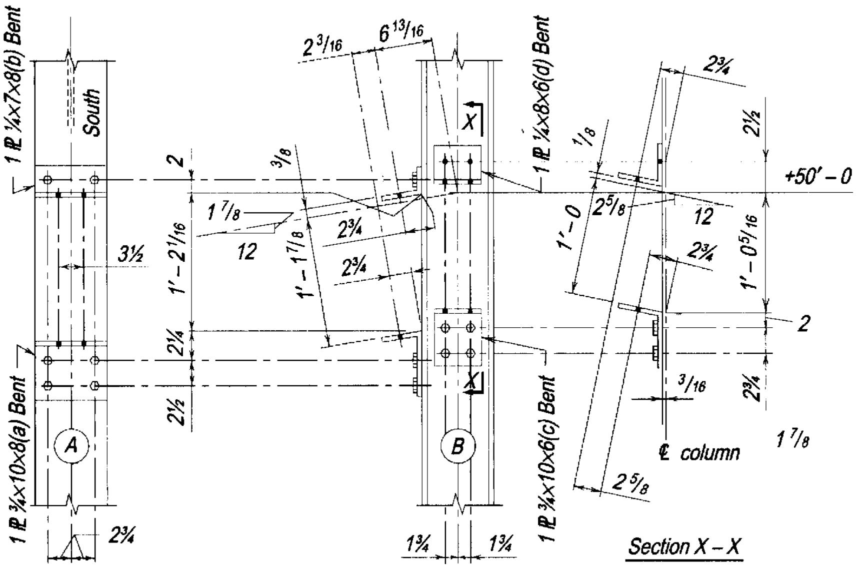


Figure 10–41. Sloped seated connections.

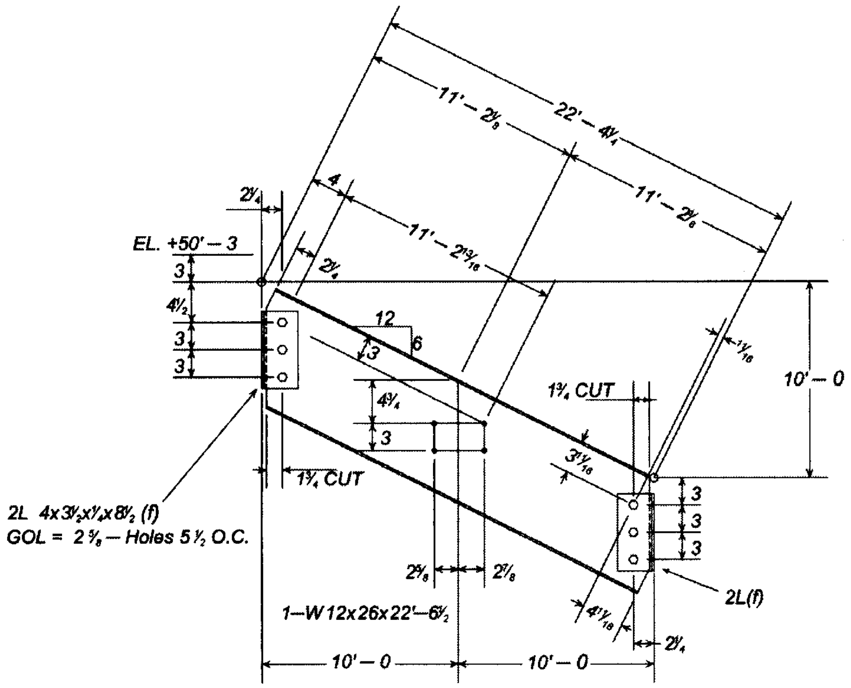


Figure 10–42. Sloped beam with rectangular connections.

As shown in Figure 10–44, the top flange of the channel and the connection angles d^R and d^L are cut to clear the flanges of beam B1. In this detail, with a 3-in-12 angle of cant, 4-in. legs were wide enough to contain the pattern of hole-punching.

Since the multiple punching or drilling of column flanges requires strict adherence to column gage lines, punching is generally skewed in the fittings. When, for some reason, this is not possible, as in Figure 10–45, skewed reference lines are shown on the column to aid in matching connections.

When canted connecting materials are assembled on the beam, particular care must be used in determining the direction of skew for punching the connection angles. An error reversing this skew may permit matching of holes in both members, but the beam will be canted opposite to the intended direction.

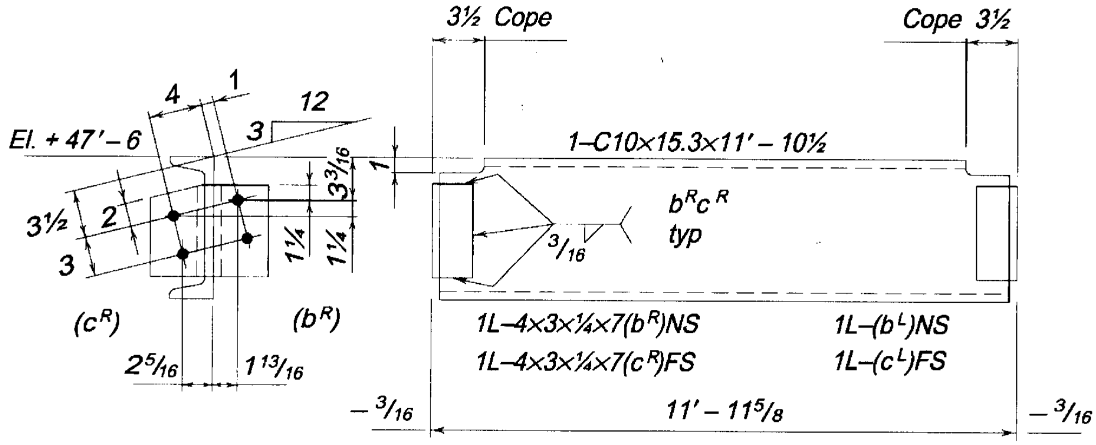


Figure 10–43. Canted double-angle connections.

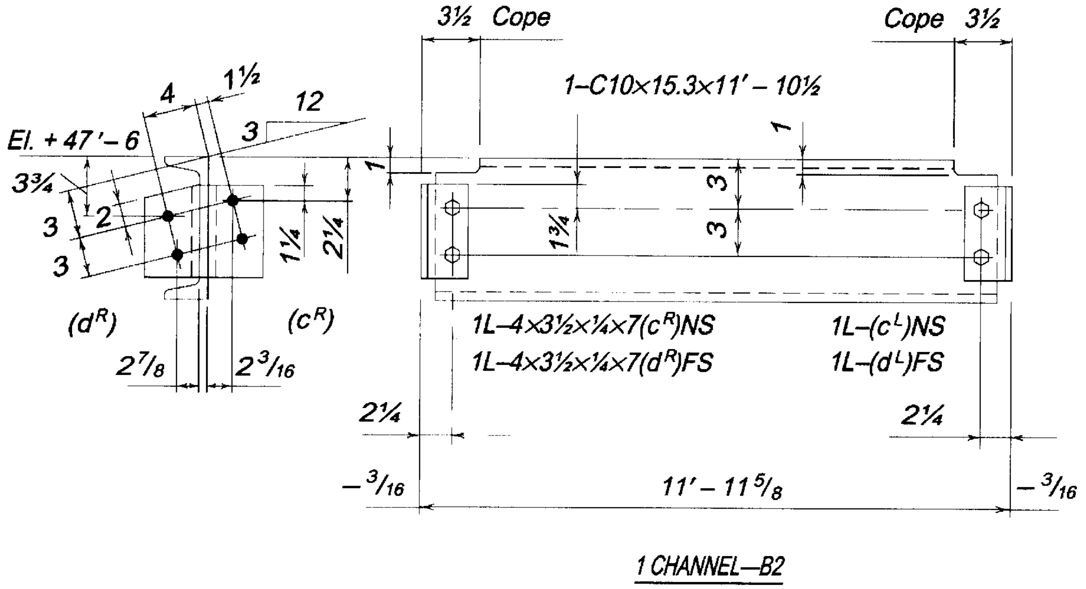


Figure 10–44. Canted connections to a sloping support.

Note the connection angles in Figure 10–45 are shown shop-welded to the beam. This was done to provide tightening clearance for ³⁄₄-in. high-strength field bolts in the opposite leg. Had the shop fasteners been bolts, it would have been necessary to stagger the field and shop fasteners and provide longer angles for the increased spacing.

Canted seated beams, shown in Figure 10–46, present few problems other than those in ordinary square-end seated beams. Sufficient width and length of angle leg must be provided to contain the gage line punching or drilling in the column face, as well as the off-center location of the holes matching the punching in the beam flange. The elevation of the top flange centerline and the bevel of the beam flange may be given for reference on the beam detail, although the bevel shown will not affect the fabrication.

Inclines in Two or More Directions (Hip and Valley Framing)

When a beam inclines in two or more directions with respect to the axis of its supporting member, it can be classified as a combination of those inclination directions. For example, the beam of Figure 10–33d is both skewed and sloped. Angle A shows the skew and angle B shows the slope. Note that, since the inclined beam is foreshortened in the elevation, the true angle B appears only in the auxiliary projection, Section X-X. The development of these details is quite complicated and graphical solutions to this compound angle work can be found in any textbook on descriptive geometry. Accurate dimensions may then be determined with basic trigonometry.

DESIGN CONSIDERATIONS FOR SIMPLE SHEAR CONNECTIONS TO HSS COLUMNS

Many of the familiar simple shear connections that are used to connect to wide-flange columns can be used with HSS columns. These include double and single angles, unstiffened and stiffened seats, single plates, and tee connections. One additional connection that is unique for HSS columns is the through-plate; note that this alternative is seldom required structurally and presents a significant economic penalty when a single plate connection would otherwise suffice. Variations in attachments are more limited with HSS columns since the connecting element will typically be shop-welded to the HSS and bolted

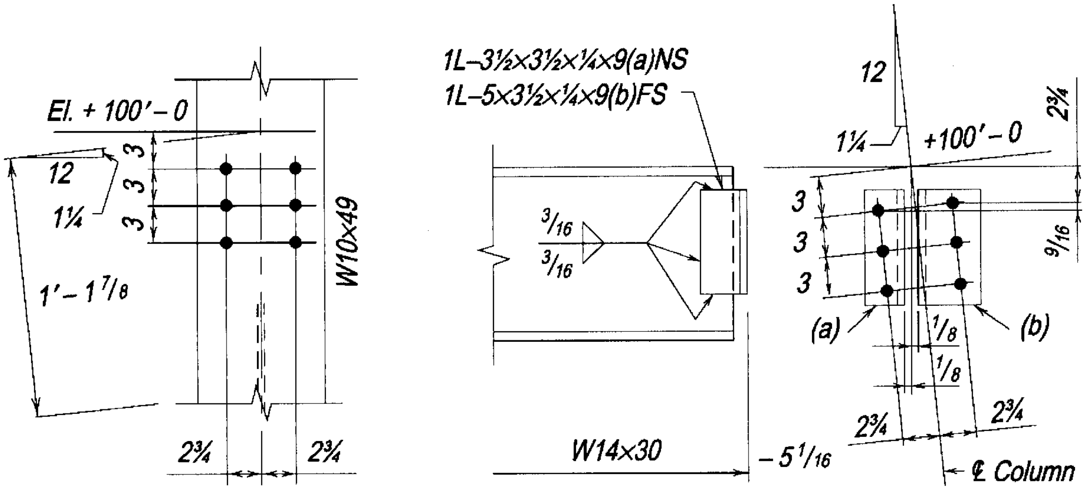


Figure 10–45. Canted connection to column flange.

to the supported beam. Except for seated connections, the bolting will be to the web of a wide-flange or other open profile section. Coping is not required except for bottom-flange copes that facilitate knifed erection with double-angle connections.

Double-Angle Connections to HSS

Table 10–1 is a design aid for double-angle connections. The table shows the compatible sizes of W-beams for the various connection configurations. Based on maximum beam web thickness, maximum weld size, maximum HSS corner radius and 4-in. outstanding angle legs, double-angle connections may be used with any HSS having a width greater than or equal to 12 in. If 3-in. outstanding angle legs are used for connections with six bolts or less, HSS with widths of 10 in. are acceptable for obtaining welds on the flat of the side. For smaller web thicknesses, welds and corner radii, it may be possible to fit the connection on widths of 10 in. if the outstanding angle legs are 4 in. and on widths of 8 in. for outstanding angle legs of 3 in. However, these dimensions must be verified for a particular case. See the tabulated workable flat dimensions for HSS in Part 1.

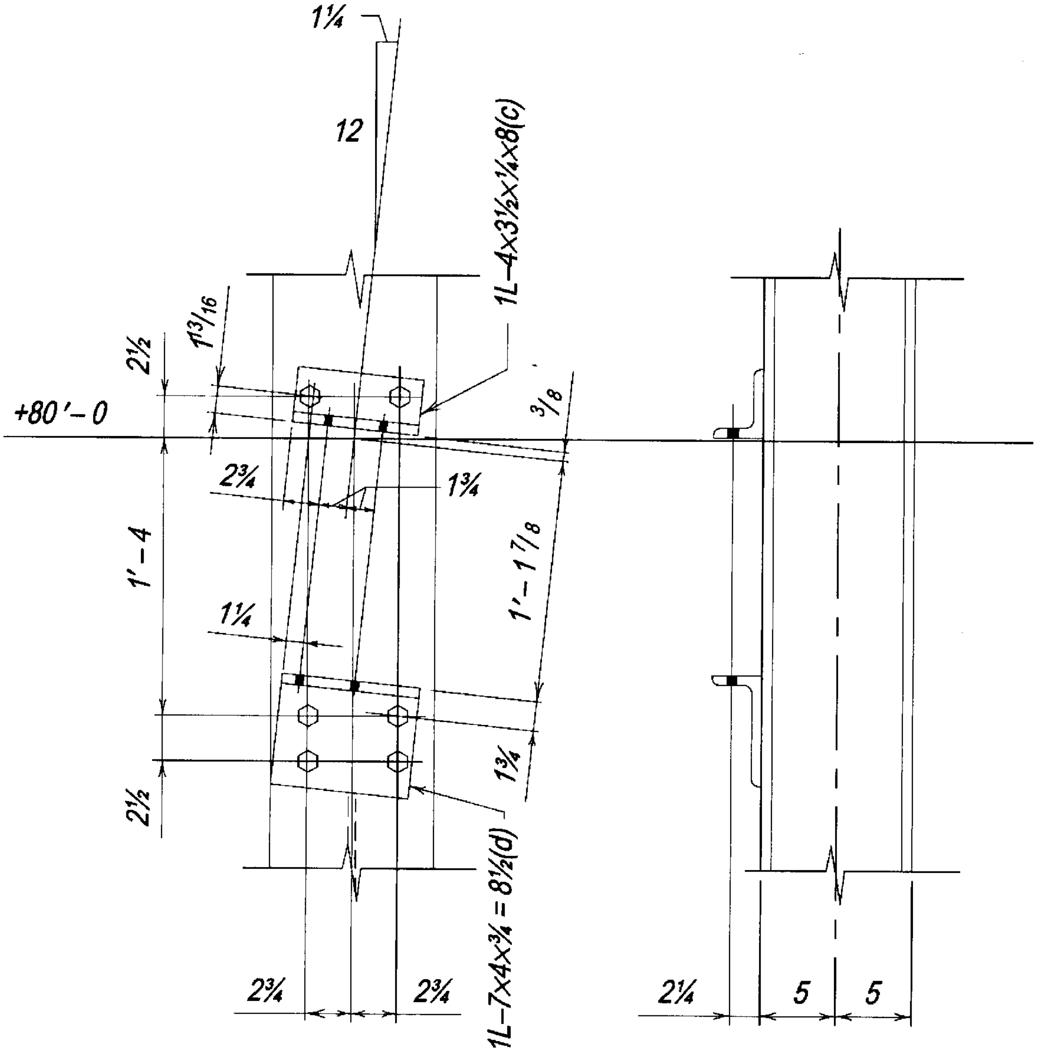


Figure 10–46. Canted seated connections.

Single-Plate Connections to HSS

As long as the HSS wall is not classified as a slender element, the local distortion caused by the single-plate connection will be insignificant in reducing the column strength of the HSS (Sherman, 1996). Therefore, single-plate connections may be used with HSS when $b/t \leq 1.40(E/F_y)^{0.5}$ or 35.1 for $F_y = 46$ ksi. Single-plate connections may also be used with round HSS as long as they are non-slender under axial load ($D/t \leq 0.11E/F_y$).

Unstiffened Seated Connections to HSS

In order to properly attach seat angles to the flat of the HSS, the workable flat must be large enough to accommodate both the width of the seat angle and the welds. Seat widths are usually 6 in. or 8 in., but other widths may also be used. See the tabulated workable flat dimensions for HSS in Part 1.

Table 10–6 may be used for unstiffened seated connections to HSS. The minimum HSS thicknesses are established based on the weld strength. If the HSS thickness is less than the minimum value, the weld strength must be reduced proportionally.

Stiffened Seated Connections to HSS

Tables 10–8 and 10–14 are design aids for stiffened seated connections. Table 10–8 is applicable to all member types, and Table 10–14 presents specific limits for HSS, based on the yield-line mechanism limit state for HSS. Some values for small connection lengths L and large HSS widths B have been reduced to meet the limit-state for a line load with a width of $0.4L$ across the HSS, per AISC Specification Section K1. The strength of the connection is obtained by multiplying the tabulated value for a particular HSS width and stiffener length by the square of the HSS thickness and dividing by the width of the seat. For combinations of B and L that are not listed in Table 10–14, the HSS does not have sufficient flat width to accommodate a weld to the seat that is $0.2L$ on each side of the stiffener. Since the required width also depends on the stiffener thickness and the HSS corner-radius, the HSS width must be checked even when the values are tabulated. See the tabulated workable flat dimensions for HSS in Part 1.

The minimum HSS thicknesses associated with the weld strengths of Table 10–8 are given in Table 10–14. If the HSS thickness is less than the minimum tabulated value, the weld strength must be reduced proportionally.

Through-Plate Connections

In the through-plate connection shown in Figure 10–47, the front and rear faces of the HSS are slotted so that the plate can be passed completely through the HSS and welded to both faces. Through-plate connections should be used when the HSS wall is classified as a slender element ($b/t > 1.40(E/F_y)^{0.5}$ or 35.1 for $F_y = 46$ ksi for rectangular HSS; $D/t > 0.11E/F_y$ for round HSS and Pipe) or does not satisfy the punching shear limit-state. A single-plate connection is more economical and should be used if the HSS is neither slender nor inadequate for the punching shear rupture limit-state.

Through-plate connections have the same limit-states as single-plate connections and Table 10–9 may be used to determine the size and number of bolts and the plate thickness. The welds, however, are subject to direct shear and may not have to be as large as those for single-plate connections. For equilibrium of the forces in Figure 10–47, the shear in the welds on the front face should not exceed the strength of the pair of welds. The HSS wall strength can be matched to the weld shear strength to determine the minimum thickness, as illustrated in Part 9. If the thickness of the HSS is less than the minimum, the weld strength must be reduced proportionally. Conservatively, the welds on the rear face may be the same size.

When a connection is made on both sides of the HSS with an extended through-plate, the portion of the plate inside the HSS is subject to a uniform bending moment. For long connections, this portion of the plate may buckle in a lateral-torsional mode prior to yielding, unless H is very small. Using a thicker plate to prevent lateral-torsional buckling would restrict the rotational flexibility of the connection. Therefore, it must be recognized that the plate may buckle and that the moment will be shared with the HSS wall in a complex manner. However, if the HSS would be satisfactory for a single-plate connection, the lateral-torsional buckling limit-state is not a critical concern involving loss of strength.

Single-Angle Connections

For fillet welding on the flat of the HSS side, while keeping the center of the beam web in line with the center of the HSS, single-angle connections must be compatible with one-half the workable flat dimension provided in Part 1. Generally, the following HSS widths and thicknesses will work:

$b = 8$ in. and $t \leq 1/4$ in.
$b = 9$ in. and $t \leq 3/8$ in.
$b \geq 10$ in. and any nominal thickness

Alternatively, single angles can be welded to narrow HSS with a flare-bevel weld.

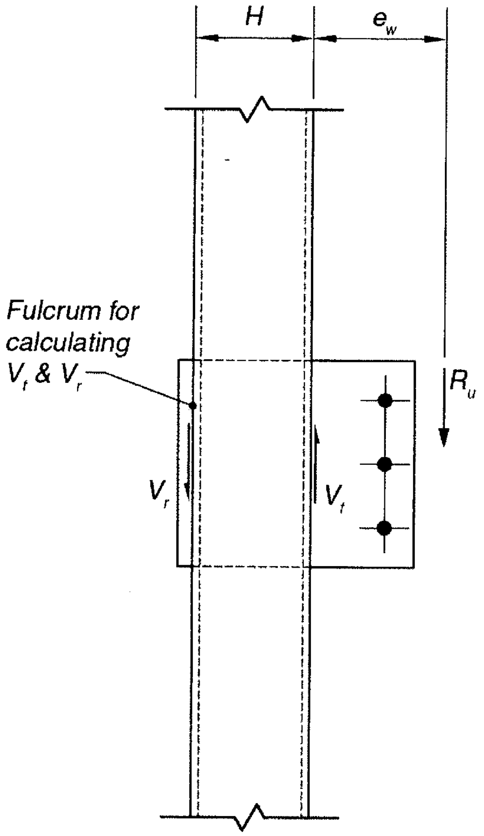


Figure 10–47. Shear forces in a through-plate connection.

Table 10–12

Minimum Inside Radius for Cold-Bending[1]

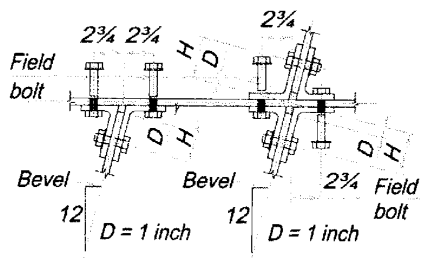
ASTM Designation[2]	Thickness t, in.			
	Up to $^3/_4$	Over $^3/_4$ to 1	Over 1 to 2	Over 2
A36, A572-42	$1^1/_2\,t$	$1^1/_2\,t$	$1^1/_2\,t$	$2t$
A242, A529-50, A529-55, A572-50, A588, A992	$1^1/_2\,t$	$1^1/_2\,t$	$2\,t$	$2^1/_2\,t$
A572-55, A852	$1^1/_2\,t$	$1^1/_2\,t$	$2^1/_2\,t$	$3\,t$
A572-60, A572-65	$1^1/_2\,t$	$1^1/_2\,t$	$3\,t$	$1^1/_2\,t$
A514	$1^3/_4\,t$	$2^1/_4\,t$	$4^1/_2\,t$	$5^1/_2\,t$

[1] Values are for bend lines perpendicular to direction of final rolling. If bend lines are parallel to final rolling direction, multiply values by 1.5.

[2] The grade designation follows the dash; where no grade is shown, all grades and/or classes are included.

Table 10–13
Clearances for All-Bolted Skewed Connections

Values given are for webs up to ¾-in. thick, angles up to ⅝-in. thick, and bent plates up to ½-in. thick. Bolts are either ⅞-in. diameter or 1 in. diameter, as noted. Values will be conservative for material thinner than the maximums listed, or for work with smaller bolts, and may be reduced to suit conditions by calculation or layout. For thicker material or larger bolts, check entering, driving, and tightening clearances and increase *D* and bolt gages as necessary. All dimensions are in inches. Enter bolts as shown.



Bent angles

Values of *H* for Various Fastener Combinations

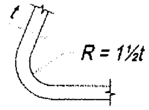
Field Bolts		⅞	1
Shop Bolts		⅞	1
Bevel	Up to 1	4*	4¼*
	Over 1 to 2	4⅛	4⅜
	Over 2 to 3	4⅜	4¾

*For back to back connections, stagger shop and field bolts or increase the 2¾-in. field bolt dimension to 3¼.

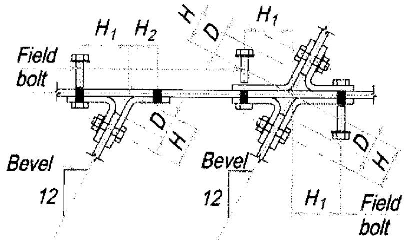
Values of *H*, *H*₁, *H*₂, and *D* for Various Bolt Combinations

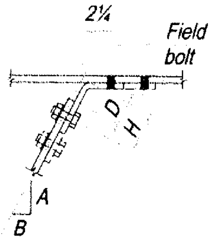
Field Fastener		⅞			1			
Shop Fastener		⅞			1			
Dimension		*H*	*H*₁	*H*₂	*H*	*H*₁	*H*₂	*D*
Bevel	Over 3 to 4	3¾	3¼	2½	4¼	3¼	2¾	1¼
	Over 4 to 5	3¾	3½	2¼	4½	3½	2½	1¼
	Over 5 to 6	4	3¾	2¼	4¾	3¾	2¼	1½
	Over 6 to 7	4½	4	2¼	5	4	2¼	1½
	Over 7 to 8	4¾	4¼	2¼	5¼	4¼	2¼	1½

Double bent plates



Min. radius of cold bend for A 36 steel up to ½ in. thick. For other bends see Table10-12





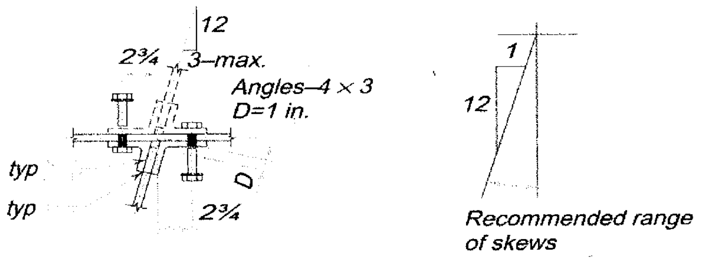
Field bolts—1 in. dia. max.
Shop bolts—1 in. dia. max.

Single bent plates

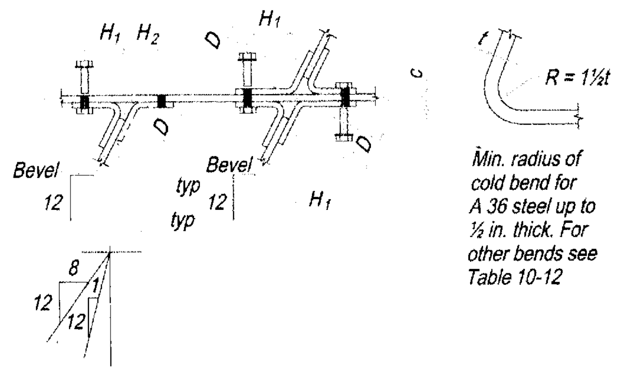
A	*B*	Shop Bolts *D*	Shop Bolts *H*
12	Over 8 to 9	1½	3
12	Over 9 to 10	1⅝	3⅛
12	Over 10 to 11	1¾	3¼
12	Over 11 to 12	1⅞	3⅜
Under 12 to 11	12	2⅛	3⅝
Under 11 to 10	12	2¼	3¾
Under 10 to 9	12	2½	4
Under 9 to 8	12	2¾	4¼
Under 8 to 7	12	3¼	4¾
Under 7 to 6	12	3¾	5¼
Under 6 to 5	12	4½	6
Under 5 to 4	12	5⅝	7⅛

Table 10–13 (continued)
Clearances for Bolted/Welded Skewed Connections

Values given are for webs up to $\frac{3}{4}$-in. thick, angles up to $\frac{5}{8}$-in. thick, and bent plates up to $\frac{1}{2}$-in. thick, with bolts 1 in. diameter maximum. Values will be conservative for thinner material and for work with smaller bolts, and may be reduced to suit conditions by calculation or layout. For thicker material or larger bolts check entering and tightening clearances and increase beam set-back D and bolt gages as necessary. Enter bolts as shown. All dimensions are in inches.



Bent angles

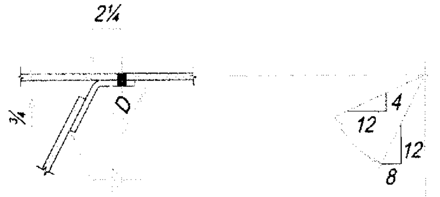


Min. radius of cold bend for A 36 steel up to ½ in. thick. For other bends see Table 10-12

Double bent plates

Bevel	D	H_1	H_2
Over 3 to 4	$c + \frac{5}{8}$	3¼	2¾
Over 4 to 5	$c + \frac{11}{16}$	3½	2½
Over 5 to 6	$c + \frac{3}{4}$	3¾	2¼
Over 6 to 7	$c + \frac{13}{16}$	4	2¼
Over 7 to 8	$c + \frac{7}{8}$	4¼	2¼

$$C = \frac{t_w}{2} + \frac{1}{16}''$$



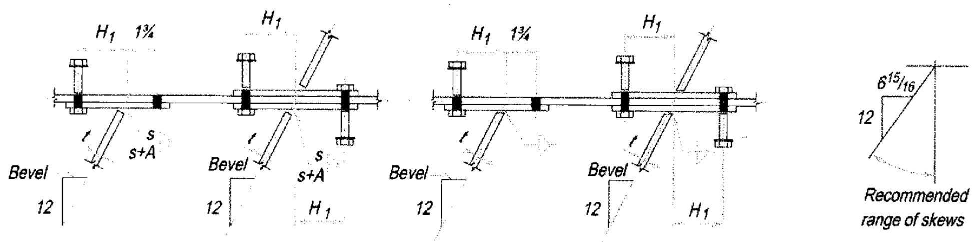
Determine value of D by calculation or layout

Recommended range of skews

Single bent plates

Table 10–13 (continued)
Clearances for Bolted/Welded Skewed Connections

Values given are for material and bolt sizes noted below. See "Shear End-Plate Connections" in Part 9 for proportioning these connections. S indicates weld size required for strength, or a size suitable to the thickness of material. When the beam web is cut square, only that portion of the table above the heavy lines is applicable. Dimension A is added to the weld size to compensate for the root opening caused by the skew. When the beam web is beveled to the required skew, values of H_1 for the entire table are valid, and A = 0. In either case, where weld strength is critical, increase the weld size to obtain the required throat dimension. Enter bolts as shown. All dimensions are in inches.



Square ends

Beveled ends

End plates

Bevel	$t = 1/4$		$t = 5/16$		$t = 3/8$		$t = 7/16$		$t = 1/2$		$t = 5/8$		$t = 3/4$	
	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A	H_1	A
Up to 1⅝	1¾	0	1¾	0	1¾	1⁄16	1¾	1⁄16	1¾	1⁄16	1⅞	⅛	1⅞	⅛
Over 1⅝ to 2⅛	1¾	0	1¾	1⁄16	1⅞	1⁄16	1⅞	1⁄16	1⅞	⅛	2	⅛	2	⅛
Over 2⅛ to 3¼	1⅞	1⁄16	1⅞	⅛	2	⅛	2	⅛	2	⅛	2⅛	0	2⅛	0
Over 3¼ to 4⅜	2⅛	⅛	2⅛	⅛	2⅛	⅛	2⅛	0	2¼	0	2¼	0	2⅜	0
Over 4⅜ to 5⅝	2¼	⅛	2¼	⅛	2⅜	0	2⅜	0	2⅜	0	2½	0	2½	0
Over 5⅝ to 6 15⁄16	2½	⅛	2½	0	2½	0	2½	0	2⅝	0	2⅝	0	2¾	0

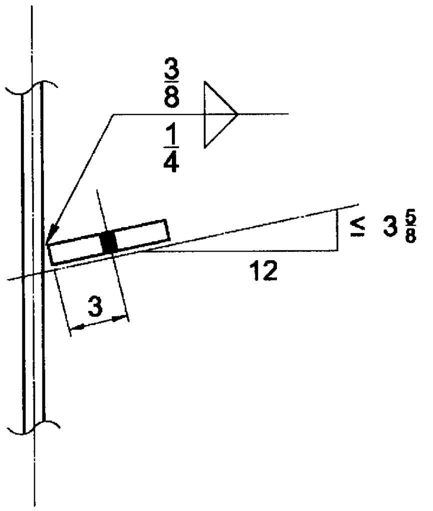
Bolts: ⅞-in. diameter maximum
End Plate thickness: ⅜-in. maximum
Supporting web thickness: ¾-in. maximum

Use of fillet welds is limited to connections with bevels of 6 15⁄16 in 12 and less.
For greater bevels consider use of double or single bent plates.

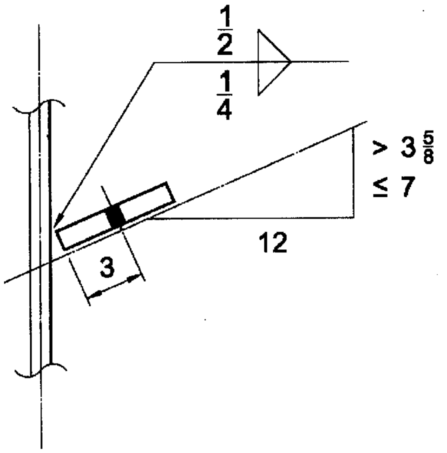
Table 10–13 (continued)
Clearances for Bolted/Welded Skewed Connections

5/16- and 3/8-in. Plate Thickness

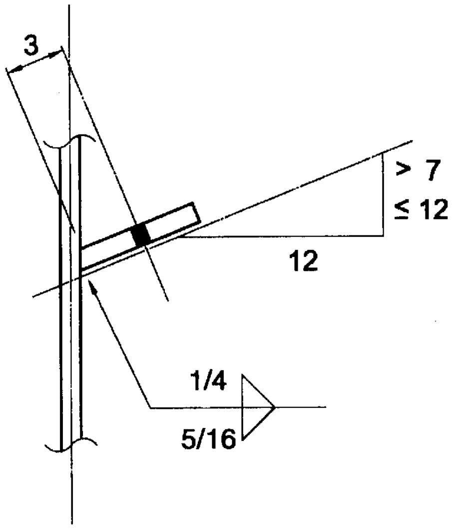
For θ ≤ 17° from Perpendicular



For 17°< θ ≤ 30° from Perpendicular



For 30°< θ ≤ 45° from Perpendicular



For θ = 45° from Perpendicular

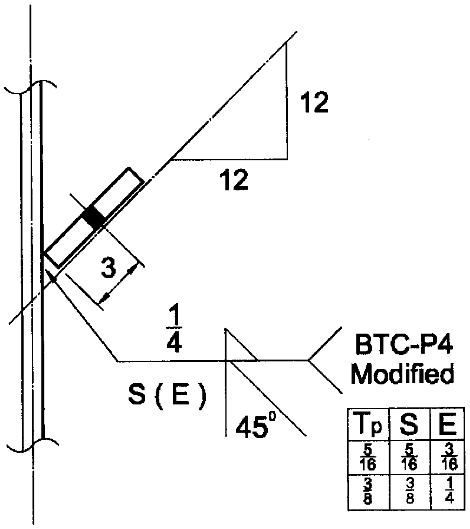
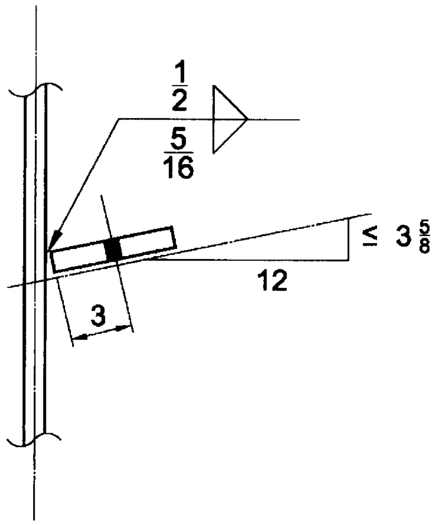


Table 10–13 (continued)

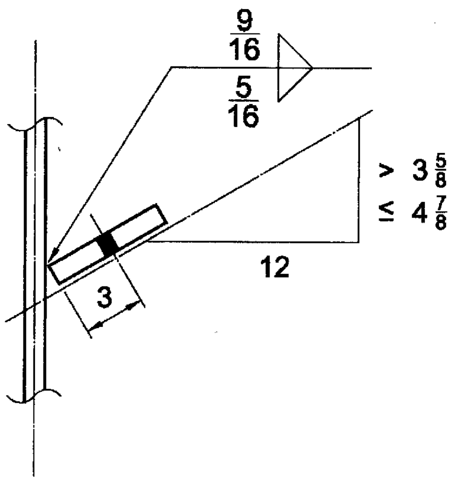
Clearances for Bolted/Welded Skewed Connections

½-in. Plate Thickness

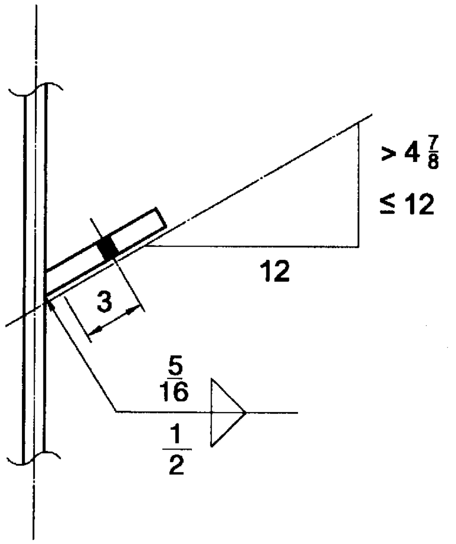
For θ ≤ 17° from Perpendicular



For 17°< θ ≤ 22° from Perpendicular



For 22°< θ ≤ 45° from Perpendicular



For θ = 45° from Perpendicular

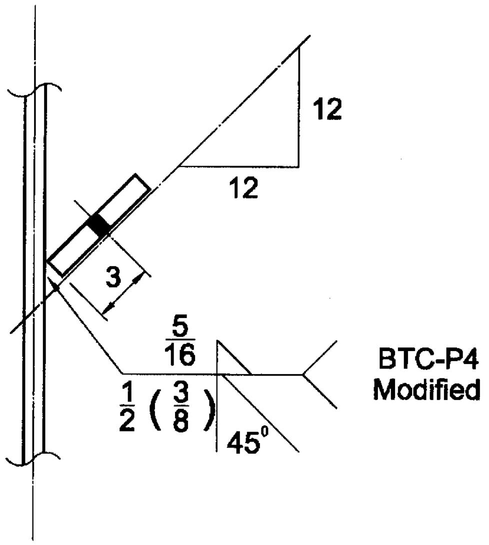


Table 10–14

Required Length and Thickness for Stiffened Seated Connections to HSS

HSS Wall Strength Factor, R_uW/t^2 or R_aW/t^2, kips/in.

L, in.	HSS Width *B*, in.											
	5		5.5		6		7		8		9	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	558	839	545	819	536	805	526	791	525	789	528	793
7	687	1030	664	997	646	971	625	940	615	925	612	920
8			798	1200	771	1160	735	1100	714	1070	704	1060
9					911	1370	856	1290	823	1240	804	1210
10					1070	1600	990	1490	942	1420	912	1370
11							1140	1710	1070	1610	1030	1550
12							1300	1960	1210	1820	1160	1740
13									1370	2060	1290	1940
14									1540	2310	1440	2170
15									1720	2580	1600	2410
16											1700	2660
17											1960	2940

Required HSS Thickness

Weld Size, in.	Min. HSS Thickness, in.
$^1/_4$	0.224
$^5/_{16}$	0.280
$^3/_8$	0.336
$^7/_{16}$	0.392
$^1/_2$	0.448
$^5/_8$	0.560

Table 10–14 (continued)

Required Length and Thickness for Stiffened Seated Connections to HSS

HSS Wall Strength Factor, R_uW/t^2 or R_aW/t^2, kips/in.

	HSS Width *B*, in.											
	10		12		14		16		18		20	
L, in.	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**	**ASD**	**LRFD**
6	**534**	802	**552**	830	**561**	843	**491**	737	**437**	656	**393**	590
7	**614**	922	**625**	940	**644**	968	**667**	1000	**594**	892	**535**	803
8	**700**	1050	**704**	1060	**717**	1080	**736**	1110	**759**	1140	**699**	1050
9	**793**	1190	**787**	1180	**794**	1190	**809**	1220	**828**	1240	**851**	1280
10	**893**	1340	**876**	1320	**876**	1320	**885**	1330	**901**	1350	**920**	1380
11	**1000**	1500	**971**	1460	**962**	1450	**965**	1450	**976**	1470	**993**	1490
12	**1120**	1680	**1070**	1610	**1050**	1580	**1050**	1580	**1060**	1590	**1070**	1600
13	**1240**	1870	**1180**	1770	**1150**	1730	**1140**	1710	**1140**	1710	**1150**	1720
14	**1370**	2070	**1290**	1940	**1250**	1880	**1230**	1850	**1220**	1840	**1230**	1840
15	**1520**	2280	**1410**	2120	**1360**	2040	**1330**	1990	**1310**	1980	**1310**	1970
16	**1670**	2510	**1540**	2320	**1470**	2210	**1430**	2150	**1410**	2120	**1400**	2100
17	**1830**	2760	**1680**	2520	**1590**	2390	**1540**	2310	**1510**	2260	**1490**	2240
18	**2010**	3020	**1820**	2740	**1710**	2570	**1650**	2470	**1610**	2420	**1590**	2380
19	**2190**	3300	**1970**	2970	**1840**	2770	**1760**	2650	**1710**	2580	**1680**	2530
20	**2390**	3600	**2130**	3210	**1980**	2980	**1880**	2830	**1820**	2740	**1790**	2680
21			**2300**	3460	**2120**	3190	**2010**	3020	**1940**	2910	**1890**	2840
22			**2480**	3730	**2280**	3420	**2140**	3220	**2060**	3090	**2000**	3010
23			**2670**	4020	**2440**	3660	**2280**	3430	**2180**	3280	**2120**	3180
24			**2870**	4310	**2600**	3910	**2430**	3650	**2310**	3480	**2230**	3360
25			**3080**	4630	**2780**	4170	**2580**	3880	**2450**	3680	**2360**	3540
26					**2960**	4450	**2740**	4110	**2590**	3890	**2480**	3730
27					**3150**	4730	**2900**	4360	**2730**	4110	**2610**	3930
28					**3350**	5030	**3070**	4620	**2880**	4330	**2750**	4130
29					**3560**	5340	**3250**	4890	**3040**	4570	**2890**	4340
30					**3770**	5660	**3440**	5160	**3200**	4810	**3040**	4560
31							**3630**	5450	**3370**	5070	**3190**	4790
32							**3830**	5750	**3540**	5330	**3340**	5020

Required HSS Thickness

Weld Size, in.	Min. HSS Thickness, in.
1/4	0.224
5/16	0.280
3/8	0.336
7/16	0.392
1/2	0.448
5/8	0.560

Table 10–14 (continued)

Required Length and Thickness for Stiffened Seated Connections to HSS

HSS Wall Strength Factor, R_uW/t^2 or R_aW/t^2, kips/in.

L, in.	HSS Width *B*, in. 22		24		26		28		30		32	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
6	357	536	328	492	302	454	281	421	262	393	246	369
7	486	730	446	669	412	618	382	574	357	535	334	502
8	635	953	582	874	537	807	499	749	466	699	437	656
9	804	1210	737	1110	680	1020	632	948	590	885	553	830
10	943	1420	910	1370	840	1260	780	1170	728	1090	682	1020
11	1010	1520	1030	1560	1020	1530	944	1420	881	1320	826	1240
12	1080	1630	1100	1660	1130	1690	1120	1690	1050	1570	983	1470
13	1160	1740	1180	1770	1200	1800	1220	1830	1230	1850	1150	1730
14	1240	1860	1250	1880	1270	1910	1290	1940	1310	1970	1330	2010
15	1320	1980	1330	2000	1340	2020	1360	2040	1380	2070	1400	2110
16	1400	2100	1410	2120	1420	2130	1430	2160	1450	2180	1470	2210
17	1490	2230	1490	2240	1500	2250	1510	2270	1530	2290	1540	2320
18	1580	2370	1570	2370	1580	2370	1590	2390	1600	2410	1620	2430
19	1670	2510	1660	2500	1660	2500	1670	2510	1680	2520	1690	2540
20	1760	2650	1750	2630	1750	2630	1750	2630	1760	2640	1770	2660
21	1860	2800	1850	2770	1840	2760	1840	2760	1840	2770	1850	2780
22	1960	2950	1940	2920	1930	2900	1920	2890	1920	2890	1930	2900
23	2070	3110	2040	3070	2020	3040	2010	3030	2010	3020	2010	3030
24	2180	3280	2140	3220	2120	3190	2110	3170	2100	3160	2100	3150
25	2290	3450	2250	3380	2220	3340	2200	3310	2190	3290	2190	3290
26	2410	3620	2360	3540	2320	3490	2300	3450	2280	3430	2280	3420
27	2530	3800	2470	3710	2430	3650	2400	3600	2380	3570	2370	3560
28	2650	3990	2590	3890	2540	3810	2500	3760	2480	3720	2460	3700
29	2780	4180	2700	4060	2650	3980	2610	3920	2580	3870	2560	3840
30	2920	4380	2830	4250	2760	4150	2710	4080	2680	4030	2650	3990
31	3050	4590	2950	4440	2880	4330	2820	4250	2780	4180	2760	4140
32	3190	4800	3080	4630	3000	4510	2940	4420	2890	4350	2860	4300

Required HSS Thickness

Weld Size, in.	Min. HSS Thickness, in.
1/4	0.224
5/16	0.280
3/8	0.336
7/16	0.392
1/2	0.448
5/8	0.560

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PART 11

DESIGN OF FLEXIBLE MOMENT CONNECTIONS

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of flexible moment connections. For the design of simple shear connections, see Part 10. For the design of fully restrained moment connections, see Part 12. For connections that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

LOAD DETERMINATION

The behavior of PR moment connections is intermediate in degree between the flexibility of simple shear connections and the full rigidity of FR moment connections. Per AISC Specification Section B3.6b, PR moment connections are permitted upon evidence that the connections to be used are capable of furnishing, as a minimum, a predictable percentage of full end restraint. For further information on the use of PR moment connections, see Geschwindner (1991), Nethercot and Chen (1988), Gerstle and Ackroyd (1989), Deierlein et al. (1990), Goverdhan (1984) and Kishi and Chen (1986).

As an alternative, flexible moment connections (FMC) can be used as a simplified and conservative approach to PR moment connection design (Geschwindner and Disque, 2005). Using FMC, any end restraint that the connection may provide to the girder is assumed zero for gravity load because of the uncertainty of that restraint after repeated loading. The beam and its web connections are thus designed as simple, considering only the gravity loads. For lateral loads, the connection is assumed to behave as an FR moment connection for analysis and the full lateral load is carried by the assigned lateral frames. The resulting flexible moment connections are then designed as "fully restrained" for the calculated required strength due to lateral loads only.

Strength

With FMC, the partially restrained connection does not achieve its final moment resisting capacity until it has been subjected to a full cycle of factored maximum specified gravity and lateral loading. This process, termed "Shakedown", is fully described in Rex and Goverdhan (2002) and Geschwindner and Disque (2005). With FMC, this shakedown moment is the plastic moment of the connection. Since the connection is assumed to provide no end restraint to the girder under gravity loads, its full capacity is available to restrain lateral load. Stressing of the connection material may occur under gravity loads but this does not reduce the final strength of the structural connection.

Stability

The stability and second order effects for FMC frames are evaluated by the familiar effective length and amplification factor methods as provided in the AISC Specification Chapter C. The effective length is calculated by the conservative assumption that, under lateral loading, the column is rotationally restrained by only one (leeward) girder, pinned at its far end. The procedure is described in detail in Geschwindner and Disque (2005) and Chen and Lui (1991).

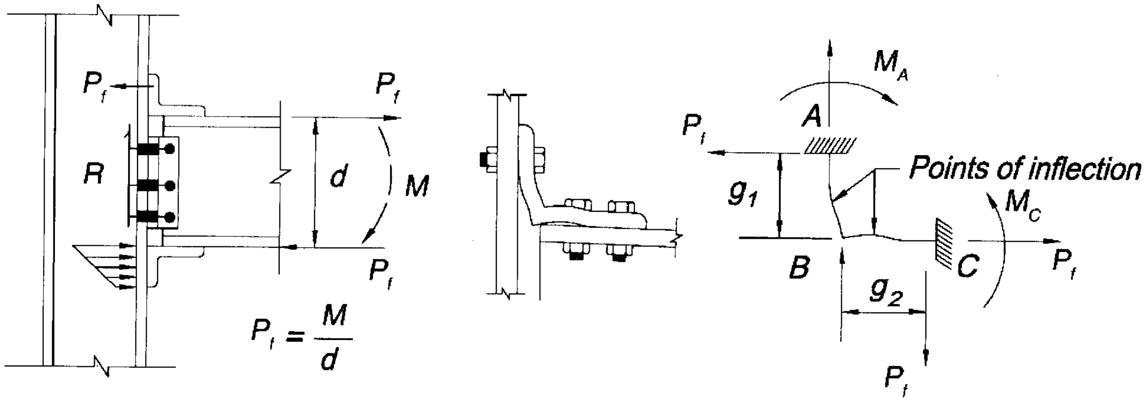
While flexible moment connections (see Figure 11–1) are not true PR moment connections, they do provide a simple, reliable and economical alternative in the design of connections that must resist lateral-load-induced moments. Flexible moment connections usually result in heavier beams and reduced rotational stiffness for columns (higher, more conservative K-factors). Additionally, there are several advantages to their use: (1) simplified analysis; (2) the beams and girders may be designed as simply connected members for gravity loads; and (3) the columns may be designed as axially loaded members with applied moments due to lateral load only. Certain provisions, however, must be met when using this type of moment connection:

1. The lateral frames must resist the lateral moments throughout the entire structure from top to bottom.
2. The beams, columns, and their connections must resist the applied moments for lateral loads.
3. The girders must be capable of carrying the full gravity load as simply supported beams.
4. The connection material must have sufficient inelastic rotation capacity to prevent the welds and/or fasteners from failing due to combined gravity and lateral loading.

Flexible moment connections deliver concentrated forces to the flanges of columns that must be accounted for in the design of the column and column panel-zone per AISC Specification Section J10. Either the column size can be selected with adequate flange and web thicknesses to eliminate the need for column stiffening, or transverse stiffeners and/or web doubler plates can be provided. For further information, refer to AISC Design Guide No. 13 *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

FLANGE-ANGLE FLEXIBLE MOMENT CONNECTIONS

Flange-angle flexible moment connections are made with top and bottom angles and a simple shear connection.



(a) (b) (c)

Figure 11–1. Flexible moment connection behavior.

The available strength of a flange-angle flexible moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

The tensile force is carried to the angle by the flange bolts, with the angle assumed to deform as illustrated in Figure 11–1. A point of inflection is assumed between the bolt gage line and the face of the connection angle, for use in calculating the local bending moment and the corresponding required angle thickness. The effect of prying action must also be considered.

The strength of this type of connection is often limited by the available angle thickness and the maximum number of fasteners that can be placed on a single gage line of the vertical leg of the connection angle at the tension flange. Figure 11–2 illustrates the column flange deformation and shows that only the fasteners closest to the column web are fully effective in transferring forces.

FLANGE-PLATED FLEXIBLE MOMENT CONNECTIONS

Originally proposed by Blodgett (1966), and illustrated in Figure 11–3, a flange-plated flexible moment connection consists of a simple shear connection and top and bottom flange plates that connect the flanges of the supported beam to the supporting column. These flange plates are welded to the supporting column and may be bolted or welded to the flanges of the supported beam. An unwelded length of 1 1/2 times the flange-plate width, b_A, is normally assumed to permit the elongation of the plate necessary for FMC behavior. Other flange-plated details are illustrated in Figures 11–4a and 11–4b.

The available strength of a flange-plated flexible moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a.

The shop and field practices for flange-plated FR moment connections (see Part 12) are equally applicable to flange-plated flexible moment connections.

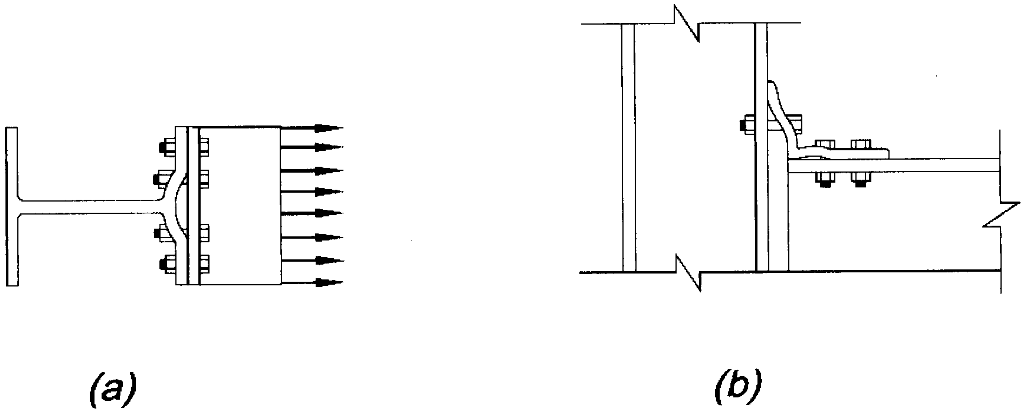
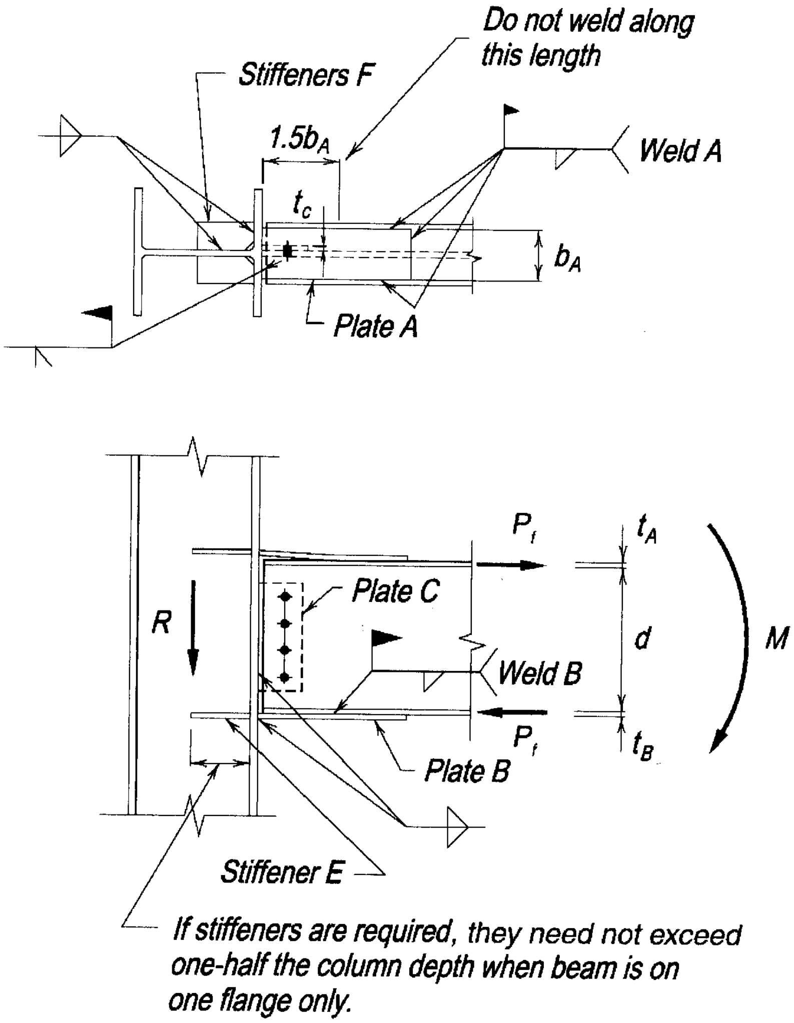


Figure 11–2. Illustration of deformations in flexible moment connections.



If stiffeners are required, they need not exceed one-half the column depth when beam is on one flange only.

Figure 11–3. Flange-plated flexible moment connection.

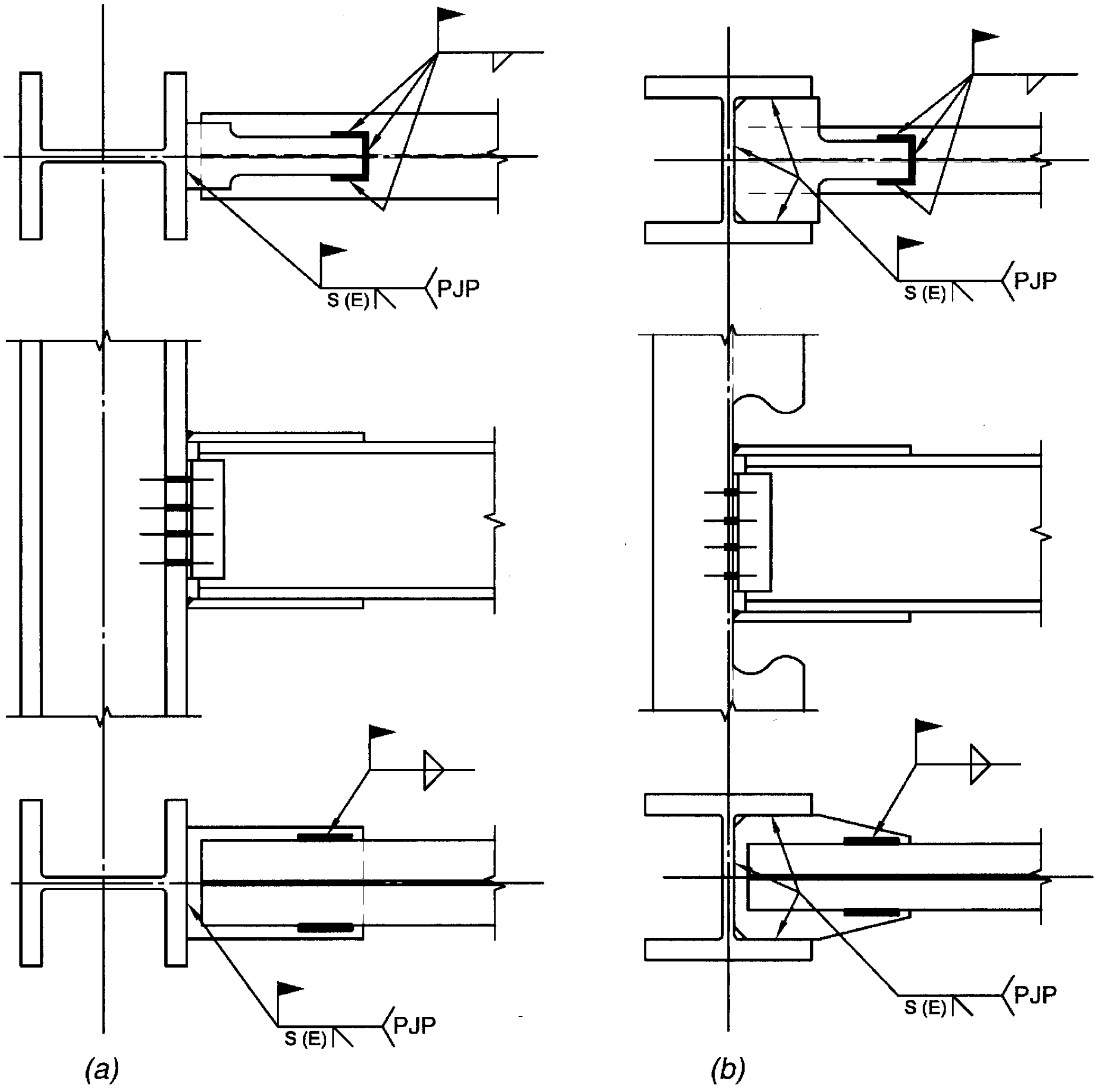


Figure 11–4. Typical flange-plated flexible moment connections.

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PART 12

DESIGN OF FULLY RESTRAINED (FR) MOMENT CONNECTIONS

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of fully restrained (FR) moment connections. For the design of simple shear connections, see Part 10. For the design of flexible moment connections, see Part 11. For FR moment connections that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

FR MOMENT CONNECTIONS

Load Determination

As defined in AISC Specification Section B3.6b, FR moment connections possess sufficient rigidity to maintain the angles between connected members at the strength limit states, as illustrated in Figure 12–1. While connections considered to be fully restrained seldom actually provide for zero rotation between members, the small amount of rotation present is usually neglected and the connection is idealized as one exhibiting zero end rotation.

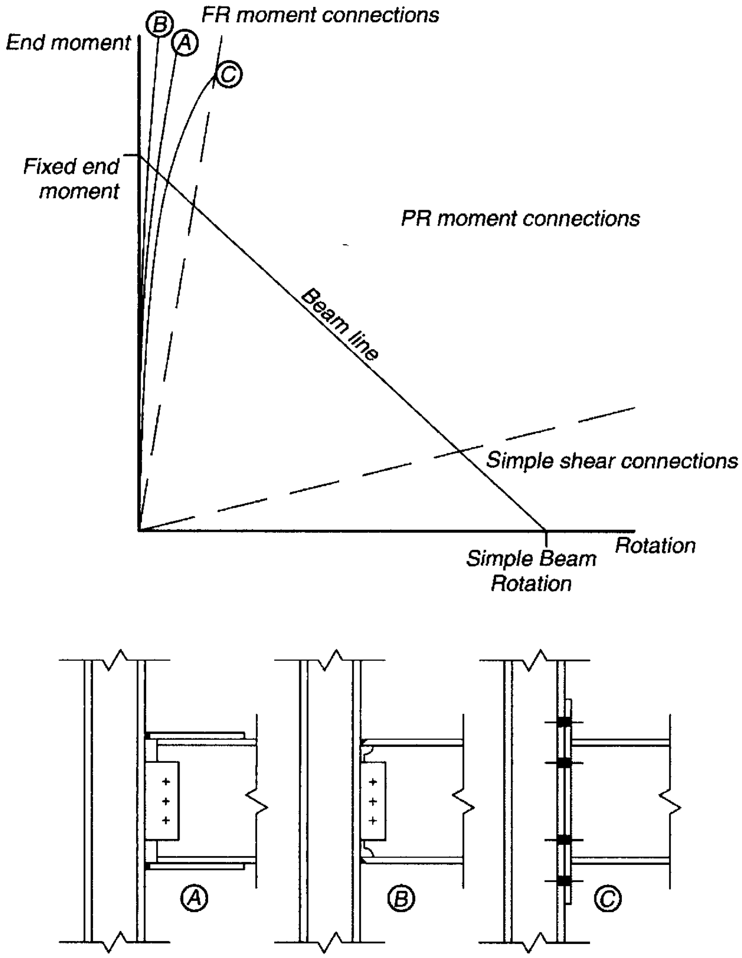


Figure 12–1. FR moment connection behavior.

End connections in FR construction are designed to carry the factored forces and moments, except that some inelastic but self-limiting deformation of a part of the connection is permitted. Huang, et al. (1973) showed that the moment can be resolved into an effective tension-compression couple acting as axial forces at the beam flanges. The flange force, P_{uf} or P_{af}, *is* determined as:

LRFD	ASD
$P_{uf} = \frac{M_u}{d_m}$	$P_{af} = \frac{M_a}{d_m}$

where

M_u or M_a = required beam end moment, kip-in.

d_m = moment arm between the flange forces, in. (varies for all FR connections and for stiffener design)

Shear is transferred through the beam-web shear connection. Since, by definition, the angle between the beam and column in an FR moment connection remains unchanged under loading, eccentricity can be neglected entirely in the shear connection. Additionally, it is permissible to use bolts in bearing in either standard or slotted holes perpendicular to the line of force. Axial forces, if present, are normally assumed to be distributed uniformly across the beam flange cross-sectional area. However, if the beam-web connection has sufficient stiffness, it can also be assumed to participate in the transfer of beam axial force.

Moment connections deliver concentrated forces to the flanges of columns that must be accounted for in the design of the column and column panel-zone per AISC Specification Section J10. Either the column size can be selected with adequate flange and web thickness to eliminate the need for column stiffening, or transverse stiffeners and/or web doubler plates can be provided. For further information, refer to AISC Design Guide No. 13 *Stiffening of Wide-Flange Columns at Moment Connections: Wind and Seismic Applications* (Carter, 1999).

Design Checks

The available strength of an FR moment connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). The effect of eccentricity in the shear connection can be neglected. Additionally, the strength of the supporting column (and thus the need for stiffening) must be checked. In all cases, the available strength, R_n/Ω or ϕR_n, must equal or exceed the required strength, R_u or R_a.

Temporary Support During Erection

Bolted construction provides a ready means to erect and temporarily connect members by use of the bolt holes. In contrast, FR moment connections in welded construction must be given special attention so that all pieces affecting the alignment of the welded joint may be erected, fitted, and supported until the necessary welds are made. Temporary support can be provided in welded construction by furnishing holes for erection bolts, temporary seats, special lugs, or by other means.

The effects of temporary erection aids on the finished structure should be considered, particularly on members subjected to tension loading or fatigue. They should be permitted to remain in place whenever possible since they seldom are reusable and the cost to remove them can be significant. If left in place, erection aids should be located so as not to cause a stress concentration. If, however, erection aids are to be removed, care should be taken so that the base metal is not damaged.

Temporary supports should be sufficient to carry any loads imposed by the erection process, such as the dead weight of the member, additional construction equipment, or material storage. Additionally, they must be flexible enough to allow plumbing of the structure, particularly in tier buildings.

Welding Considerations for Fully Restrained Moment Connections

Field welding should be arranged for welding in the flat or horizontal position and preference should be given to fillet welds over groove welds, whenever possible. Additionally, the joint detail and welding procedure should be constructed to minimize distortion and the possibility of lamellar tearing.

The typical complete-joint-penetration groove weld in a directly welded flange connection for a rolled beam can be expected to shrink about $^1/_{16}$ in. in the length dimension of the beam when it cools and contracts. Thicker welds, such as for welded plate-girder flanges, will shrink even more—up to $^1/_8$ in. or $^3/_{16}$ in. This amount of shrinkage can cause erection problems in locating and plumbing the columns along lines of continuous beams. A method of calculating weld shrinkage can be found in Lincoln Electric Co. (1973). Unnecessarily thick stiffeners with complete-joint-penetration groove welds should be avoided since the accompanying weld shrinkage may contribute to lamellar tearing.

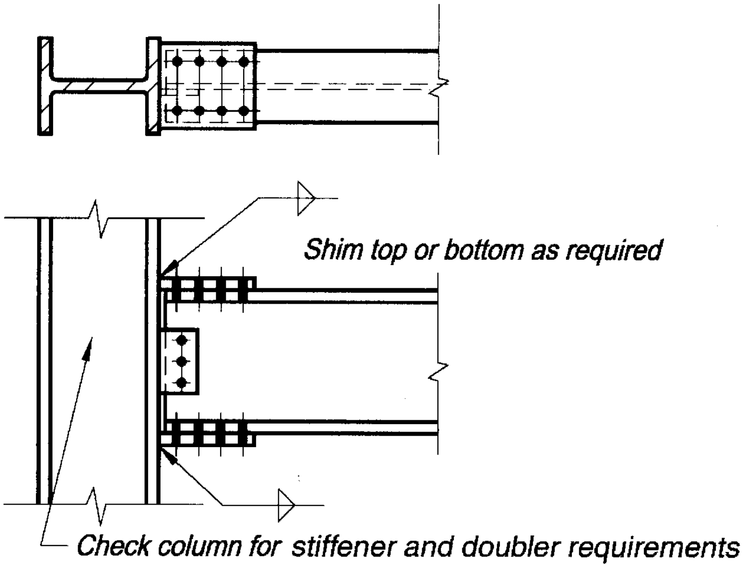
Weld shrinkage can best be controlled by fabricating the beam longer than required by the amount of the anticipated weld shrinkage. Alternatively, the weld-joint root opening can be increased. For further information, refer to AWS D1.1.

FR CONNECTIONS WITH WIDE-FLANGE COLUMNS

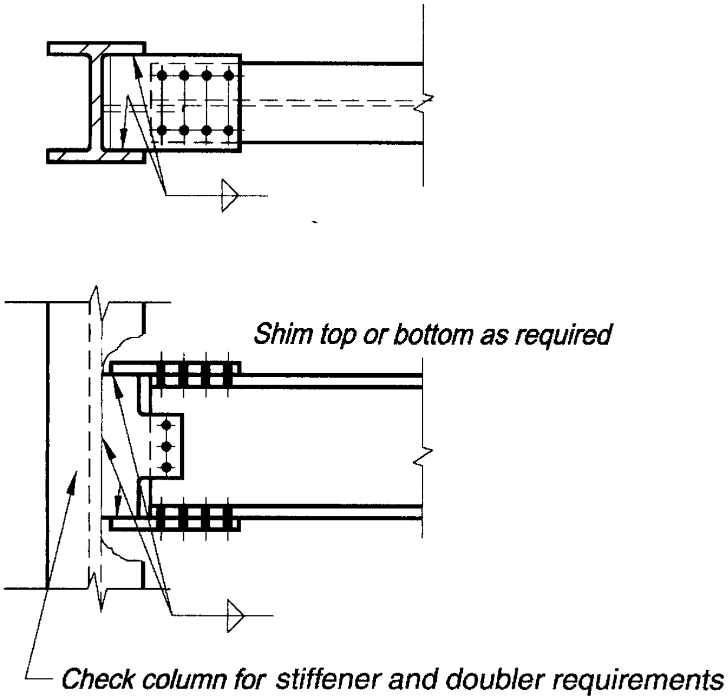
Flange-Plated FR Moment Connections

As illustrated in Figure 12–2, a flange-plated FR moment connection consists of a shear connection and top and bottom flange plates that connect the flanges of the supported beam to the supporting column. These flange plates are welded to the supporting column and may be bolted or welded to the flanges of the supported beam.

In a column-flange connection, the flange plates are usually located with respect to the column web centerline. Because of the column-flange mill tolerance on out-of-squareness with the web, it is desirable to shop-fit long flange plates from the theoretical column-web centerline to assure good field fit-up with the beam. Misalignment on short connections, as illustrated in Figure 12–3, can be accommodated by providing oversized holes in the plates. Since mill tolerances in both the beam and the column may cause significant shop and/or field assembly problems, it may be desirable to ship the flange plates loose for field attachment to the column.

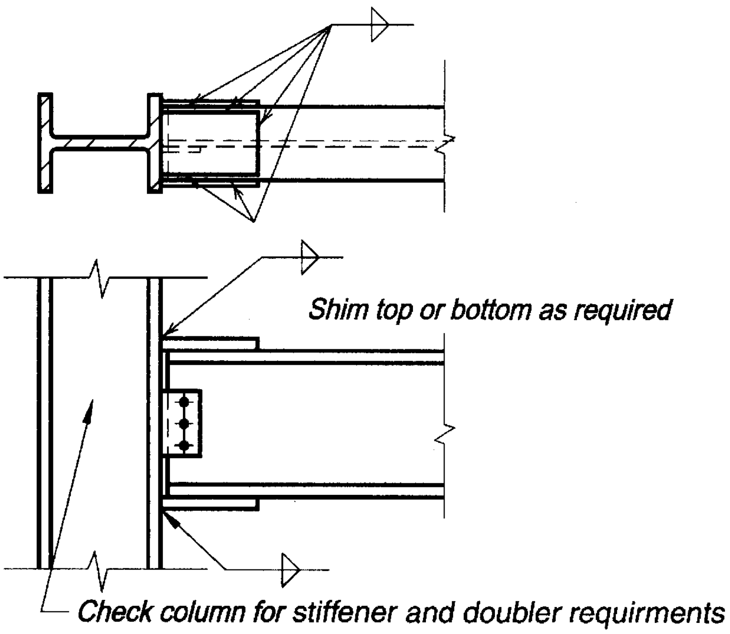


(a) Column flange support, bolted flange plates



(b) Column web support, bolted flange plates

Figure 12–2. Flange-plated FR moment connections.



(c) Column flange support, welded flange plates

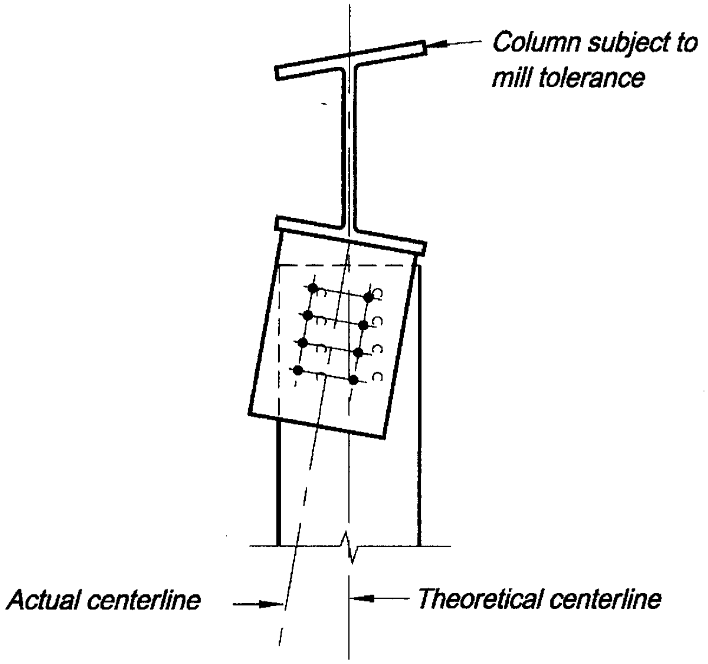
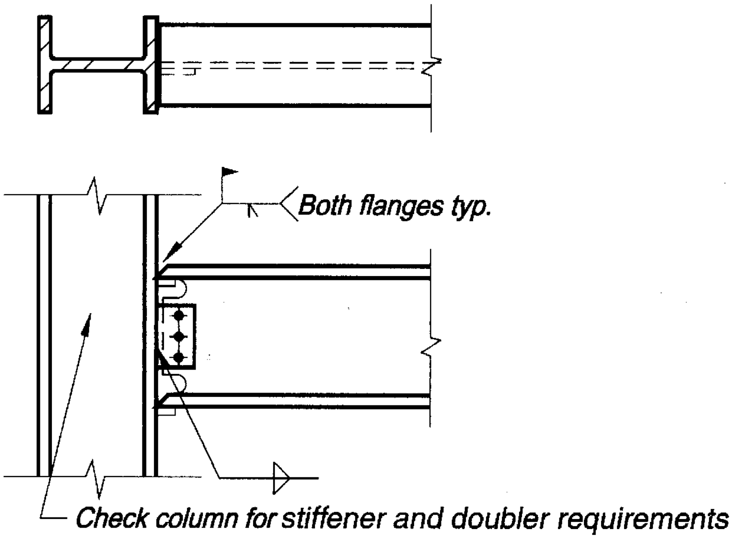


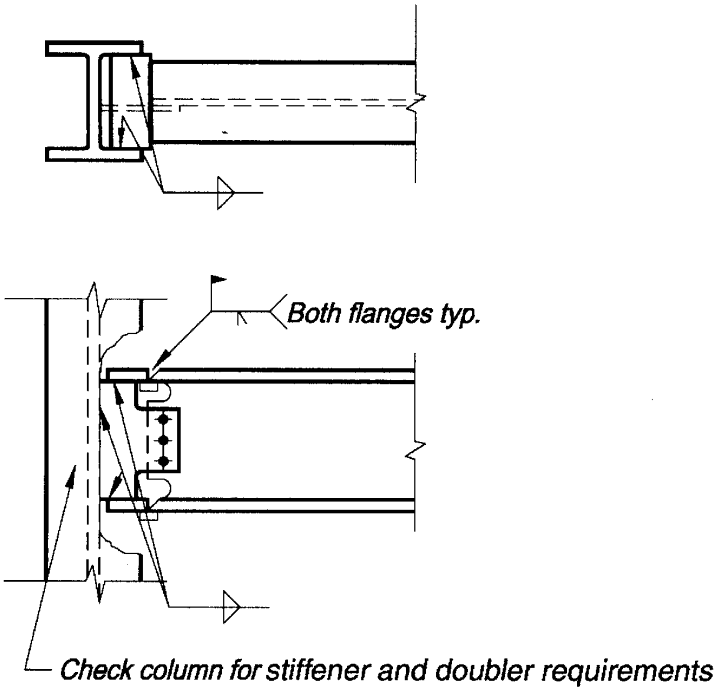
Figure 12–3. Effect of mill tolerances on flange-plated connections.

Directly Welded Flange FR Moment Connections

As illustrated in Figure 12–4, a directly welded flange FR moment connection consists of a shear connection and complete-joint-penetration groove welds, which directly connect the top and bottom flanges of the supported beam to the supporting column. Note, in Figure 12–4b, the stiffener extends beyond the toe of the column flange to eliminate the effects of triaxial stresses.



(a) Column flange support



(b) Column web support

Figure 12–4. Directly welded flange FR moment connections.

Extended End-Plate FR Moment Connections

As illustrated in Figure 12–5, an extended end-plate moment connection consists of a plate of length greater than the beam depth, perpendicular to the longitudinal axis of the supported beam. The end-plate is always welded to the web and flanges of the supported beam and bolted to the supporting member. The principal advantage of extended end-plate moment connections is that all welding is done in the shop. Thus, the erection process is relatively fast and economical.

Figure 12–6 illustrates three commonly used extended end-plate connections. The connections are classified by the number of bolts at the tension flange and by the presence of end-plate to beam flange stiffeners. The four-bolt unstiffened and stiffened extended end-plate connections of Figure 12–6a and 12–6b are generally limited by bolt strength. The connection is compatible for use with nearly one-half of the available beam sections.

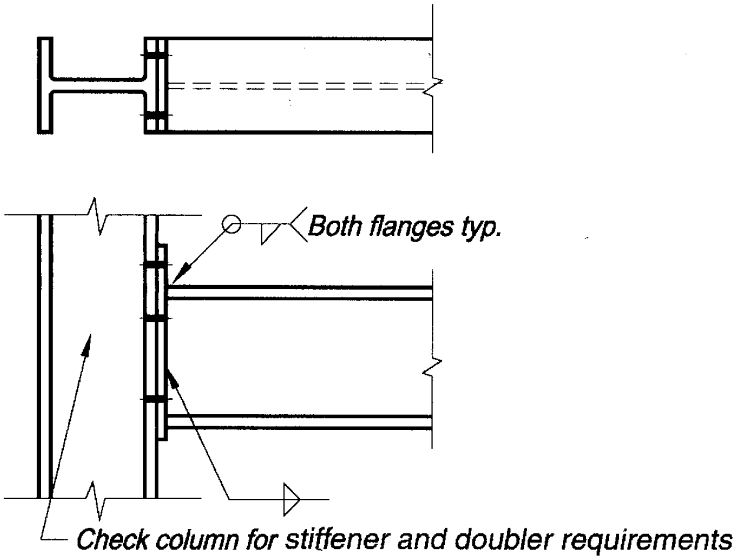


Figure 12–5. Extended end-plate FR moment connection.

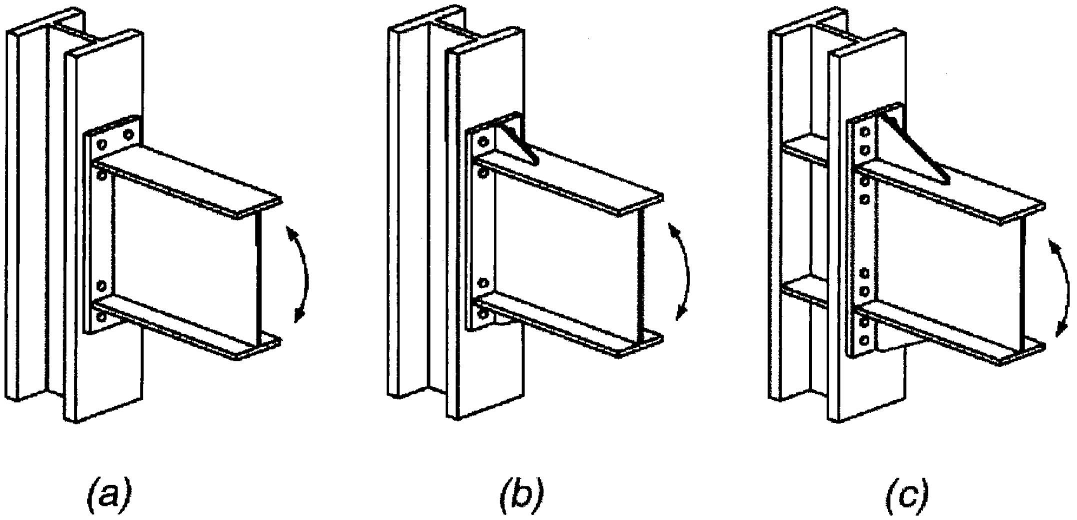


Figure 12–6. Configurations of extended end-plate FR moment connections.

Alternatively, the eight-bolt stiffened extended end-plate connection shown in Figure 12–11c is generally compatible with approximately 90 percent of the available beam sections.

Complete discussion of the design procedures, along with design examples, are found in AISC Design Guide Series No. 4, 2nd Ed., *Extended End-Plate Moment Connections – Seismic and Wind Applications* (Murray and Sumner, 2003). Design procedures and example calculations for nine other end-plate connections, which are commonly used in the metal building industry, are found in AISC Design Guide Series No. 16, *Flush and Extended Multiple Row Moment End-Plate Connections* (Murray and Shoemaker, 2002). Recommended shop and field erection practices, basic design assumptions, and a brief overview of the design procedures follow.

Shop and Field Practices

End-plate moment connections require extra care in shop fabrication and field erection. The fit-up of extended end-plate connections is sensitive to the column flange conditions and may be affected by column flange-to-web squareness, beam camber, or squareness of the beam end. The beam is frequently fabricated short to accommodate the column overrun tolerances with shims furnished to fill any gaps which might result.

As reported by Murray and Meng (1996), use of weld access holes can result in beam flange cracking. If CJP welds are used, the weld cannot be inspected over the web; however, because this location is a relative "soft" spot in the connection, it is of no concern.

Design Assumptions

A summary of the assumptions made in the design guide procedures follows:

1. ASTM A325 or A490 high-strength bolts of diameter not greater than 1½ in. must be used.
2. The minimum specified yield stress of the end-plate material must be less than 50 ksi.
3. When the procedures in AISC Design Guide 16 are used, only static loading is permitted (wind, seismic with seismic response modification factor, R, taken equal to or less than 3, snow, and temperature loads are considered static loads).
4. The recommended minimum distance from the face of the beam flange to the nearest bolt centerline (the vertical bolt pitch) is the bolt diameter d_b plus ½ in. if the bolt diameter is not greater than 1 in., and ¾ in. for larger diameter bolts. However, many fabricators prefer to us a standard pitch dimension of 2 in. or 2½ in. for all bolt diameters.
5. All of the shear force at a connection is assumed to be resisted by the compression side bolts. End-plate connections need not be designed as slip-critical connections and it is noted that shear is rarely a major concern in the design of moment end-plate connections.
6. The end-plate width effective in resisting the applied moment must be taken as not greater than the beam flange width b_f plus 1 in.
7. The gage of the tension bolts (horizontal distance between vertical bolt lines) must not exceed the beam tension flange width.
8. When CJP welds are used, weld access holes should not be used, and the weld between beam flange-to-web fillets should treated as a PJP weld.
9. For non-seismic connections, when the required resisting moment is less than the

available flexural strength of the beam, the end-plate connection can be designed for required moment but not less than 60 percent of the beam strength.

10. Beam web-to-end-plate welds in the vicinity of the tension bolts should be designed to develop the yield stress of the beam web unless the required moment is less than 60 percent of the beam flexural strength.
11. Only the web-to-end-plate weld between the mid-depth of the beam and the inside face of the beam compression flange or the weld between the inner row of tension bolts plus $2d_b$ and the inside face of the beam compression flange, whichever is smaller, is considered effective in resisting the beam end shear.

Design Procedures

The design procedure in AISC Design Guide 4, 2nd Ed., and AISC Design Guide 16 differ from those in previous AISC design methods. The new procedures are based on yield-line analysis for determining end-plate thickness and modified tee-hanger analysis to determine required bolt strength. The procedures in AISC Design Guide 4, 2nd Ed., are for pretensioned bolts and "thick plates", and result in connections with the smallest possible bolt diameter. For these connections, prying forces are zero. The procedures in AISC Design Guide 16 allow for both "thick plate" and "thin plate" designs. A thin plate design results in the smallest possible end-plate thickness and the maximum bolt prying force. In addition, connections can be designed using either pretensioned or snug-tight bolts.

Column side design procedures are included in AISC Design Guide 4, 2nd Ed. Both Design Guides have complete examples for the various end-plate configurations.

FR MOMENT SPLICES

Beams and girders sometimes are spliced in locations where both shear and moment must be transferred across the splice. Per AISC Specification Section J6, the nominal strength of the smaller section being spliced must be developed in groove-welded butt splices. Other types of beam or girder splices must develop the strength required by the actual forces at the point of the splice.

Location of Moment Splices

A careful analysis is particularly important in continuous structures where a splice may be located at or near the point of inflection. Since this inflection point can and does migrate under service loading, actual forces and moments may differ significantly from those assumed. Furthermore, since loading application and frequency can change in the lifetime of the structure, it is prudent for the designer to specify some minimum strength requirement at the splice. Hart and Milek (1965) propose that splices in fixed-ended beams be located at the one-sixth point of the span and be adequate to resist a moment equal to one-sixth of the flexural strength of the member, as a minimum.

Force Transfer in Moment Splices

Force transfer in moment splices can be assumed to occur in a manner similar to that developed for FR moment connections. That is, the shear, R_u or R_a, is primarily transferred through the beam-web connection and the moment can be resolved into an effective tension-compression couple where the required force at each flange, P_{uf} or P_{af}, is determined by:

LRFD	ASD
$P_{uf} = \frac{M_u}{d_m}$	$P_{af} = \frac{M_a}{d_m}$

where

M_u or M_a = required moment in the beam at the splice, kip-in.

d_m = moment arm, in. (varies based upon actual connection geometry)

Axial forces, if present, are normally assumed to be distributed uniformly across the beam flange cross-sectional area. However, if the beam-web connection has sufficient stiffness, it can also be assumed to participate in the transfer of beam axial force.

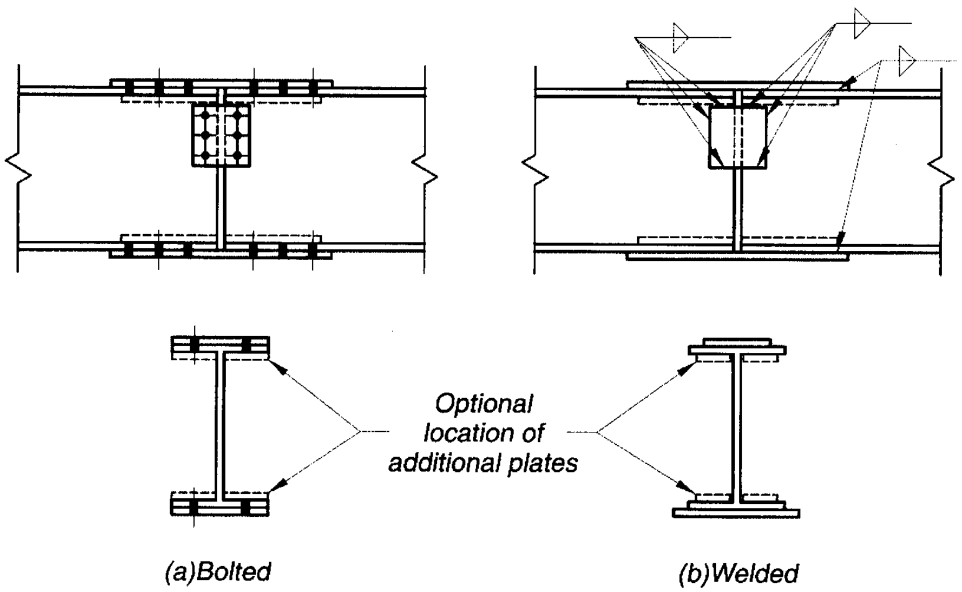
Flange-Plated FR Moment Splices

Moment splices can be designed as shown in Figure 12–7, to utilize flange plates and a web connection. The flange plates and web connection may be bolted or welded.

The splice and spliced beams should be checked in a manner similar to that described previously under "Flange-Plated FR Moment Connections," except that the web connection should be designed as illustrated previously for shear splices in Part 10 without consideration of eccentricity.

Figure 12–7 illustrates two types of splices, bolted and welded. Figure 12–7a illustrates the detail of a bolted flange-plated moment splice. For this case, the flange plates are normally made approximately the same width as the beam flange as shown in Figure 12–7a.

Alternatively, Figure 12–7b illustrates the detail of a welded splice. As shown in Figure 12–7b, the top plate is narrower and the bottom plate is wider than the beam flange, permitting the deposition of weld metal in the downhand or horizontal position without inverting the beam. While this is a benefit in shop fabrication (the beam does not have to be turned over), it is of extreme importance in the field where the weld can be made in the



(a)Bolted (b)Welded

Figure 12–7. Flange-plated moment splice.

horizontal instead of the overhead position, since the beam cannot be turned over. This detail also provides tolerance for field alignment, since the joint gap can be opened or closed. When splices are field-welded, some means for temporary support must be provided as discussed previously in "Temporary Support During Erection".

If the beam or girder flange is thick and the flange forces are large, it may be desirable to place additional plates on the insides of the flanges. In a bolted splice (Figure 12–7a), the bolts are then loaded in double shear and a more compact joint may result. Note that these additional plates must have sufficient area to develop their share of the double-shear bolt load.

In a welded splice (Figure 12–7b), these additional plates must have sufficient area to match the strength of the welds that connect them. Additionally, these plates must be set away from the beam web a distance sufficient to permit deposition of weld metal as shown in Figure 12–8a. This distance is a function of the beam depth and flange width, as well as the welding equipment to be used. A distance of 2 to $2^1/2$ in. or more may be required for this access. One alternative is to bevel the bottom edge of the plate to clear the beam fillet and place the plate tight to the beam web with a fillet weld as illustrated in Figure 12–8b. The effects of this bevel on the area of the plate must be considered in determining the required plate width and thickness. Another alternative would be to use unbeveled inclined plates as shown in Figure 12–8b.

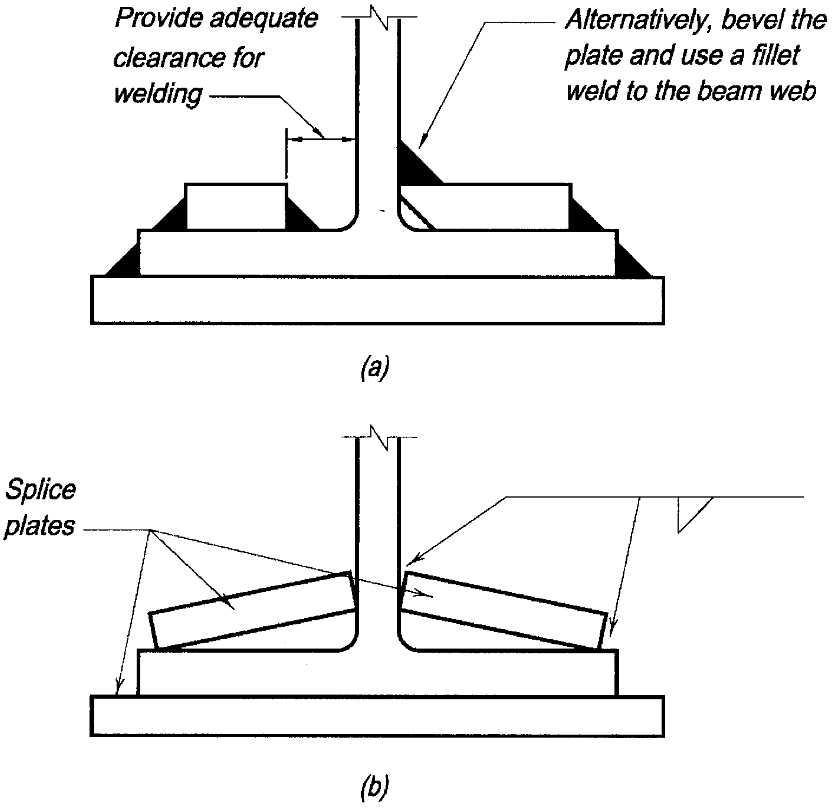


Figure 12–8. Welding clearances for flange-plated moment splices.

Directly Welded Flange FR Moment Splices

Moment splices can be designed, as shown in Figure 12–9, to utilize a complete-joint-penetration groove weld connecting the flanges of the members being spliced. The web connection may then be bolted or welded.

The splice and spliced beams should be checked in a manner similar to that described previously under "Directly Welded Flange FR Moment Connections," except that the web connection should be designed as illustrated previously for shear splices in Part 10.

Although rare in occurrence, some spliced members must be level on top. Where the depths of these spliced members differ, consideration should be given to the use of a flange plate of uniform thickness for the full length of the shallower member. This avoids the fabrication problems created by an inverted transition.

In Figure 12–10, the spliced shapes are different sizes, but of the same shape depth grouping. Because rolled shapes from the same shape depth grouping have the same dimension between the flanges, aligning the inside flange surfaces avoids a more difficult offset transition. Eccentricity resulting from differing flange thicknesses is usually ignored in the design. The web plates normally are aligned to their center lines and the 1 in $2^1/2$ slope is chamfered into the flange or the weld is sloped, depending upon the relative thicknesses.

The groove- (butt-) welded splice preparation shown in Figure 12–9 may be used for either shop or field welding. Alternatively, for shop welding where the beam may be turned over, the joint preparation of the bottom flange could be inverted.

In splices subjected to dynamic or fatigue loading, the backing bar should be removed and the weld should be ground flush when it is normal to the applied stress (AISC, 1977). The access holes should be free of notches and should provide a smooth transition at the juncture of the web and flange.

Extended End-Plate FR Moment Splices

Moment splices can be designed as shown in Figure 12–11 where the tension force is in the bottom flange, to utilize four-bolt unstiffened extended end-plates connecting the members being spliced. If the end-plate and the bolts are designed properly, it is possible to load this type of connection to reach the full plastic moment capacity of the beam, $\phi_b M_p$ or M_p/Ω.

The splice and spliced beams should be checked in a manner similar to that described previously under "Extended End-Plate FR Moment Connections."

The comments for "Extended End-Plate Connections" are equally applicable to extended end-plate moment splices.

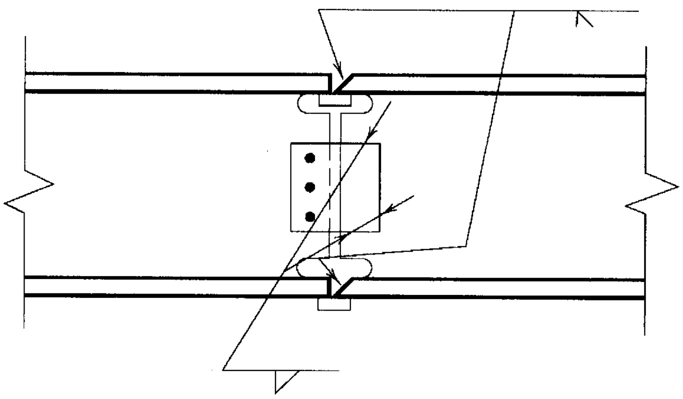


Figure 12–9. Directly welded flange moment splice.

SPECIAL CONSIDERATIONS

FR Moment Connections to Column Webs

It is frequently required that FR moment connections be made to column webs. While the mechanics of analysis and design do not differ from FR moment connection to column flanges, the details of the connection design as well as the ductility considerations required are significantly different.

Recommended Details

When an FR moment connection is made to a column web, it is normal practice to stop the beam short and locate all bolts outside of the column flanges as illustrated in Figure 12–2. This simplifies the erection of the beam and permits the use of an impact wrench to tighten all bolts. It is also preferable to locate welds outside the column flanges to provide adequate clearance.

Ductility Considerations

Driscoll and Beedle (1982) discuss the testing and failure of two FR moment connections to column webs: a directly welded flange connection and a bolted flange-plated connection, shown respectively in Figures 12–12a and 12–12b. Although the connections in these tests were proportioned to be "critical," they were expected to provide inelastic rotations at full plastic load. Failure occurred unexpectedly, however, on the first cycle of loading; brittle fracture occurred in the tension connection plate at the load corresponding to the plastic moment before significant inelastic rotation had occurred.

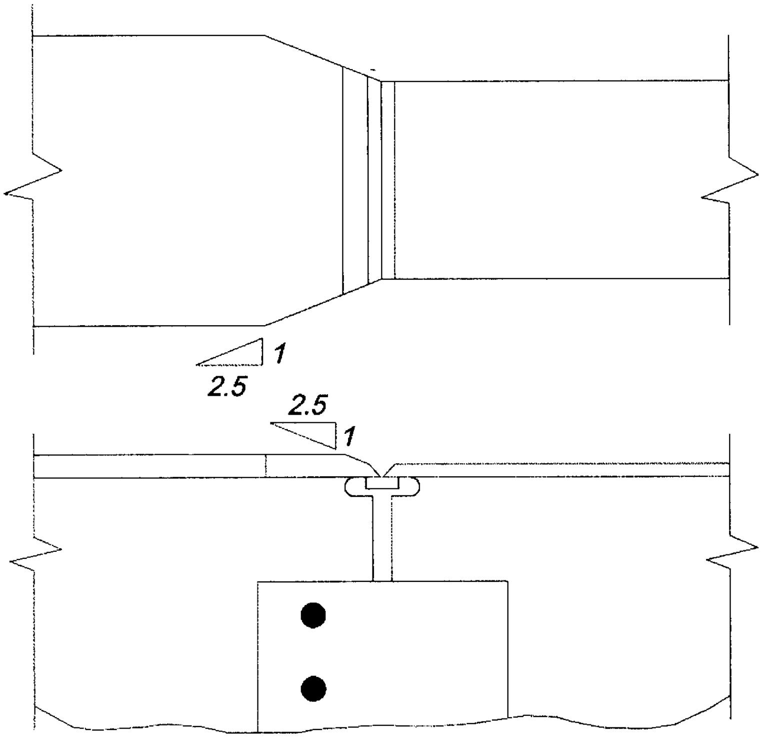


Figure 12–10. Transitions at tension flange for directly welded flange moment splices, when required.

Examination and testing after the unexpected failure revealed that the welds were of proper size and quality and that the plate had normal strength and ductility. The following is quoted, with minor editorial changes relative to figure numbers, directly from Driscoll and Beedle (1982).

> "Calculations indicate that the failures occurred due to high strain concentrations. These concentrations are: (1) at the junction of the connection plate and the column flange tip and (2) at the edge of the butt weld joining the beam flange and the connection plate."
>
> "Figure 12–13 illustrates the distribution of longitudinal stress across the width of the connection plate and the concentration of stress in the plate at the column flange tips. It also illustrates the uniform longitudinal stress distribution in the connection plate at

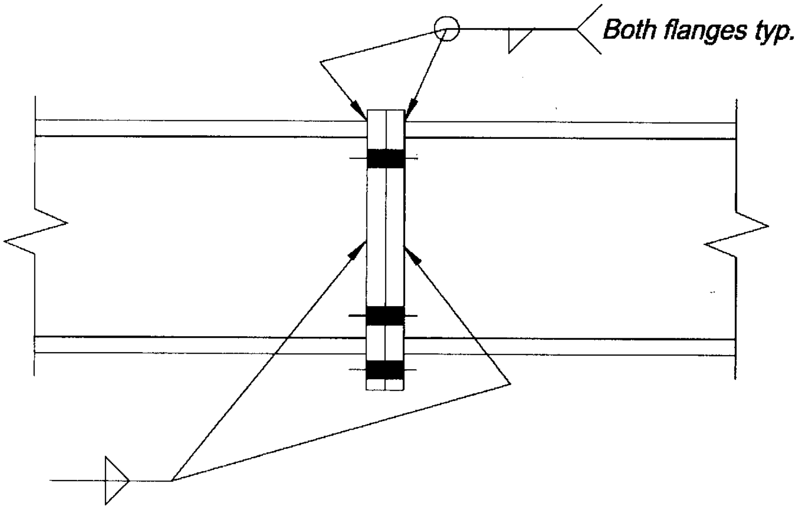
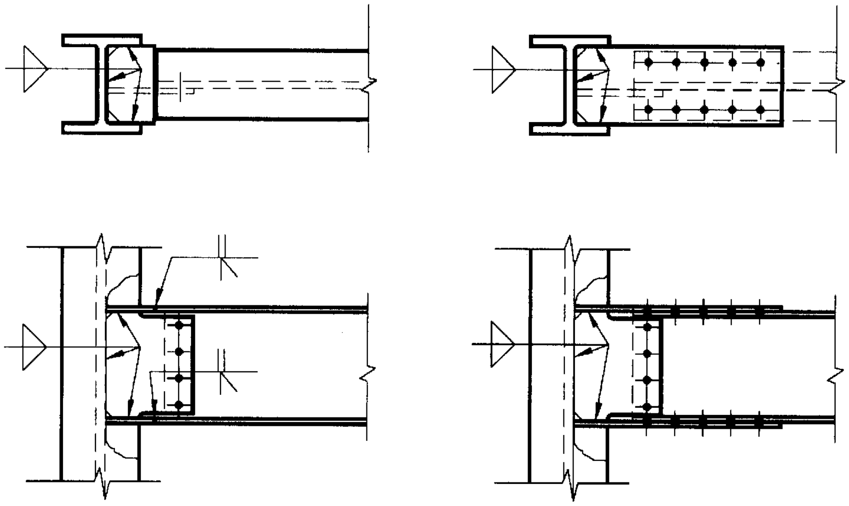


Figure 12–11. Extended end-plate moment splice.


(a) Directly welded flange FR connection

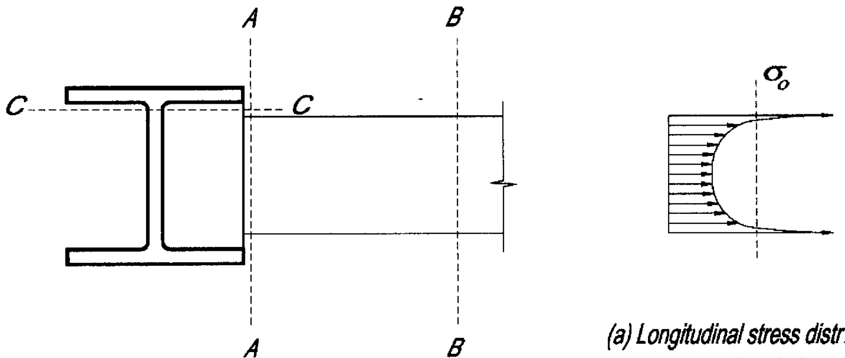
(b) Bolted flange-plated FR connection

Figure 12–12. Test specimens used by Driscoll and Beedle (1982).

some distance away from the connection. The stress distribution shown represents schematically the values measured during the load tests and those obtained from finite element analysis. (σ_o is a nominal stress in the elastic range.) The results of the analyses are valid up to the loading that causes the combined stress to equal the yield point. Furthermore, the analyses indicate that localized yielding could begin when the applied uniform stress is less than one-third of the yield point. Another contribution of the non-uniformity is the fact that there is no back-up stiffener. This means that the welds to the web near its center are not fully effective."

"The longitudinal stresses in the moment connection plate introduce strains in the transverse and the through-thickness directions (the Poisson effect). Because of the attachment of the connection plate to the column flanges, restraint is introduced; this causes tensile stresses in the transverse and the through-thickness directions. Thus, referring to Figure 12–13, tri-axial tensile stresses are present along Section A-A and they are at their maximum values at the intersections of Sections A-A and C-C. In such a situation, and when the magnitudes of the stresses are sufficiently high, materials that are otherwise ductile may fail by premature brittle fracture."

The results of nine simulated weak-axis FR moment connection tests performed by Driscoll, et. al. (1983) are summarized in Figure 12–14. In these tests, the beam flange was simulated by a plate measuring either 1 in.×10 in. or 1 1/8 in.×9 in. The fracture strength exceeds the yield strength in every case, and sufficient ductility is provided in all cases except for that of Specimen D. Also, if the rolling direction in the first five specimens (A, B, C, D, and E) were parallel to the loading direction, which would more closely approximate an actual beam flange, the ductility ratios for these would be higher. The



(a) Longitudinal stress distribution on Section A-A

(b) Longitudinal stress distribution on Section B-B

(c) Shear stress distribution on Section C-C

Figure 12–13. Stress distributions in test specimens used by Driscoll and Beedle (1982).

connections with extended connection plates (i.e., projection of 3 in.), with extensions either rectangular or tapered, appeared equally suitable for the static loads of the tests.

Based on the tests, Driscoll, et. al. (1983) report that those specimens with extended connection plates have better toughness and ductility and are preferred in design for seismic loads, even though the other connection types (except D) may be deemed adequate to meet the requirements of many design situations.

In accordance with the preceding discussion, the following suggestions are made regarding the design of this type of connection:

1. For directly welded (butt) flange-to-plate connections, the connection plate should be thicker than the beam flange. This greater area accounts for shear lag and also provides for misalignment tolerances.

   AWS D1.1, Section 3.3.3 restricts the misalignment of abutting parts such as this to 10 percent of the thickness, with 1/8-in. maximum for a part restrained against bending due to eccentricity of alignment. Considering the various tolerances in mill rolling (± 1/8 in. for W-shapes), fabrication, and erection, it is prudent design to call for the connection plate thickness to be increased to accommodate these tolerances and avoid the subsequent problems encountered at erection. An increase of 1/8 in. to 1/4 in. generally is used.

   Frequently, this connection plate also serves as the stiffener for a strong axis FR or PR moment connection. The welds that attach the plate/stiffener to the column flange may then be subjected to combined tensile and shearing or compression and shearing forces. Vector analysis is commonly used to determine weld size and stress.

   It is good practice to use fillet welds whenever possible. Welds should not be made in the column *k*-area for strength.
2. The connection plate should extend at least 3/4 in. beyond the column flange to avoid intersecting welds and to provide for strain elongation of the plate. The extension should also provide adequate room for runout bars when required.
3. Tapering an extended connection plate is only necessary when the connection plate is not welded to the column web (Specimen E, Figure 12–14). Tapering is not necessary if the flange force is always compressive (e.g., at the bottom flange of a cantilevered beam).
4. To provide for increased ductility under seismic loading, a tapered connection plate should extend 3 in. Alternatively, a backup stiffener and an untapered connection plate with 3-in. extension could be used.

Normal and acceptable quality of workmanship for connections involving gravity and wind loading in building construction would tolerate the following:

1. Runoff bars and backing bars may be left for beam with flange thicknesses greater than 2 in. (subject to tensile stress only) where they are welded to columns or used as tension members in a truss.
2. Welds need not be ground, except as required for nondestructive testing.
3. Connection plates that are made thicker or wider for control of tolerances, tensile stress, and shear lag need not be ground or cut to a transition thickness or width to match the beam flange to which they connect.
4. Connection plate edges may be sheared, or plasma- or gas-cut.

5. Intersections and transitions may be made without fillets or radii.
6. Flame-cut edges may have reasonable roughness and notches within AWS tolerances.

If a structure is subjected to loads other than gravity and wind loads, such as seismic, dynamic, or fatigue loading, more stringent control of the quality of fabrication and erection with regard to stress risers, notches, transition geometry, welding, and testing may be necessary; refer to the AISC Seismic Provisions.

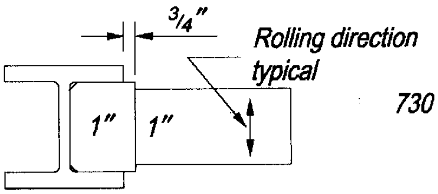
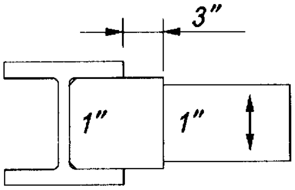
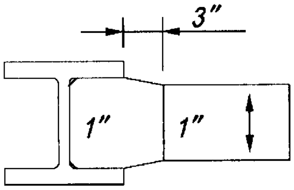
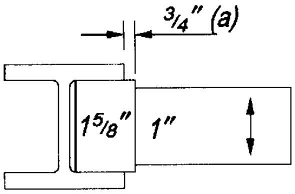
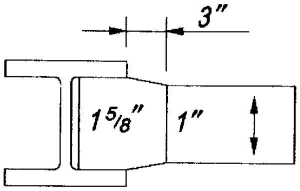
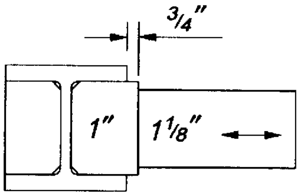
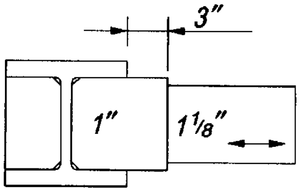
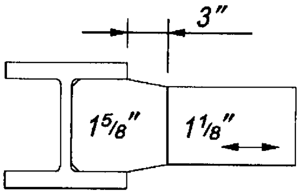
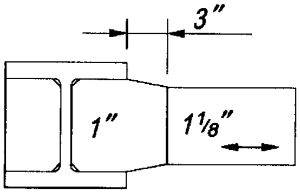
Specimen No.	Sketch W14x257 (typical)	Fracture Load (kips)	Fracture Load / Yield Load	Ductility Ratio
A		730	1.38	6.3
B		824	1.55	5.3
C		756	1.43	5.43
D		570	1.11	1.71

Figure 12–14. Results of weak-axis FR moment connection ductility tests performed by Driscoll, et al. (1983).

Specimen No.	Sketch W14x257 (typical)	Fracture Load (kips)	Fracture Load / Yield Load	Ductility Ratio
E		802	1.51	6.81
A2		762	1.40	17.7
B2		795	1.46	16.5
E2		814	1.49	16.4[(b)]
C2		813	1.49	29.6

Notes: (a) 3/4″ dimension is estimated—no dimension given.

(b) Ductility ratio estimated. Actual value not known due to malfunction in deflection gage.

Figure 12–14. (continued)

FR Moment Connections Across Girder Supports

Frequently, beam-to-girder-web connections must be made continuous across a girder-web support, as with continuous beams and with cantilevered beams at wall, roof-canopy, or building lines. While the same principles of force transfer discussed previously for FR moment connections may be applied, the designer must carefully investigate the relative stiffness of the assembled members being subjected to moment or torsion and provide the fabricator and erector with reliable camber ordinates.

Additionally, the design should still provide some means for final field adjustment to accommodate the accumulated tolerances of mill production, fabrication, and erection; it is very desirable that the details of field connections provide for some adjustment during erection. Figure 12–15 illustrates several details that have been used in this type of connection and the designer may select the desirable components of one or more of the sketches to suit a particular application. Therefore, these components are discussed here as a top flange, bottom flange, and web connection.

Top Flange Connection

As shown in Figure 12–15a, the top flange connection may be directly welded to the top flange of the supporting girder. Figures 12–15b and 12–15c illustrate an independent splice plate that ties the two beams together by use of a longitudinal fillet weld or bolts. This tie plate does not require attachment to the girder flange, although it is sometimes so connected to control noise if the connection is subjected to vibration.

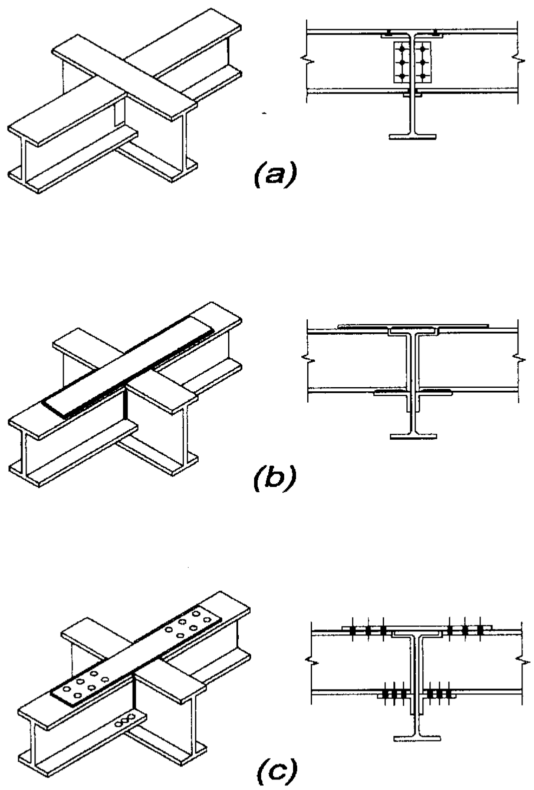


Figure 12–15. FR moment connections across girder-web supports.

Bottom Flange Connection

When the bottom flanges deliver a compressive force only, the flange forces are frequently developed by directly welding these flanges to the girder web as illustrated in Figure 12–15a. Figure 12–15b illustrates the use of an angle or channel below the beam flange to provide for a horizontal fillet weld. The angle or channel should be wider than the beam flange to allow for downhand welding. Figure 12–15c is similar, but uses bolts instead of welds to develop the flange force.

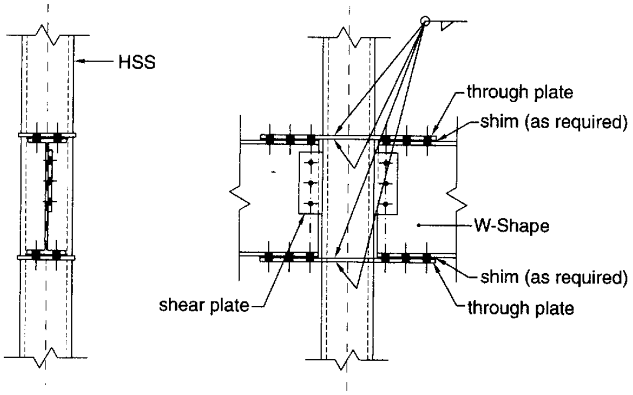
Web Connection

While a single-plate connection is shown in Figure 12–15a and unstiffened seated connections are shown in Figures 12–15b and 12–15c, any of the shear connections in Part 10 may be used. Note that the effect of eccentricity in the shear connection may be neglected.

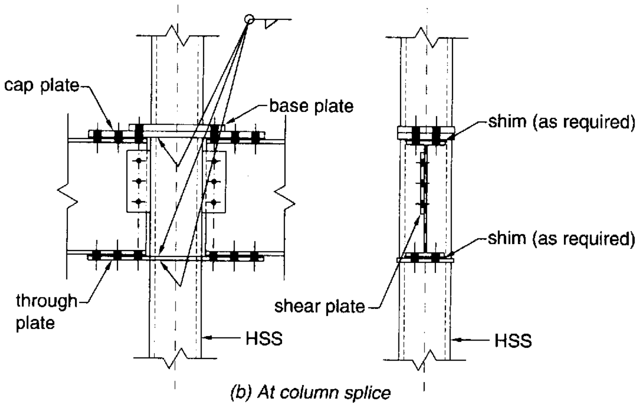
FR CONNECTIONS WITH HSS

HSS Through-Plate Flange-Plated FR Moment Connections

If the required moment transfer to the column is larger than can be provided by the bolted base plate or cap plate, or if the HSS width is larger than that of the wide flange beam, a through-plate moment connection can be used as illustrated in Figure 12–16. It should be noted that through-plate connections are more difficult to erect than the continuous beam connected framing.



(a) Between column splices



(b) At column splice

Figure 12–16. Through-plate moment connection.

When moment connections are made using through-plates such as is shown in Figure 12–16, the fabricator must allow adequate clearance between the through-plates and the structural section W-shape so as to allow for the combined effects of mill, fabrication, and erection tolerances. The permissible mill tolerances for W-shape variations in depth and squareness are shown in Table 1–22. Shimming in the field during erection with conventional shims or finger shims is the most commonly used method to fill the gap between the W-shape and the through-plate.

Specific design considerations for through-plate moment connections are as follows:

1. In Figures 12–16a and 12–16b, the column moment transfer into the joint is limited by the fillet weld of the HSS to the through-plates. If necessary, a partial-joint-penetration (PJP) groove weld can be used to improve the connection strength or a complete-joint-penetration (CJP) groove weld with backup bars can be used.
2. In Figure 12–16 an end plate (base plate) is employed to create a splice in the column. Bolt tension with prying on the base plate will determine its thickness and thus limit the moment that can be transferred to the upper HSS.
3. The cap plate, which is also a flange splice plate, should be at least the same thickness as the base plate so that moment transfer between the HSS columns need not rely on load transfer through the beam flanges. The cap plate may need to be thicker than the HSS base plate due to the combined effect of plate bending from the bolted base plate and plate tension or compression from the wide flange moment transfer.
4. The welding of the HSS to the cap and through-plate must be examined for both the HSS normal forces and the shear produced from the moment transfer from the W-shape.

HSS Cut-out Plate Flange-Plated FR Moment Connections

An alternative to interrupting the HSS for the cover or through plate is to use a wider plate with a cut-out to slip around the HSS as illustrated in Figure 12–17. A shear plate can be placed on the front and rear of the HSS faces to provide simple connections for perpendicular beams. The cut-out plate can easily be extended on the near and far sides so that a moment splice is created about both horizontal axes through the joint. The perpendicular framing should ideally be of the same depth for this detail to work well or, in the case of the simple connections, the perpendicular beams could be shallower than the space between the horizontal plates. The cut-out plates are shown as shop-welded; however, they could be field-welded.

For cut-out plate connections, the erection of the beams is more difficult than for continuous beam connections. The beams must be slipped between the two plates and against the single plate connection with shimming being required, unless the upper plate is field-welded in place.

Design Considerations for HSS Directly Welded FR Moment Connections

It may be possible to accomplish the moment transfer to the HSS without having to use a WT splice plate, end-plates, or diaphragm plates. Significant moment transfer can be achieved by attaching the W-shape directly to the face of the HSS either by welding, or by bolting. These connections are capable of developing the available flexural strength of the HSS. The available flexural strength of the W-shape, however, is seldom achieved because of the flexibility of the HSS wall.

The flexural strength for the welded W-shape is based on the strength of the respective flanges in tension and compression acting against the face of the HSS. This flange force can be considered to be the same as that of a plate with the dimensions of the flange.

Several limit states exist for the plate length (flange width) oriented perpendicular to the length of the HSS (Packer and Henderson, 1992).

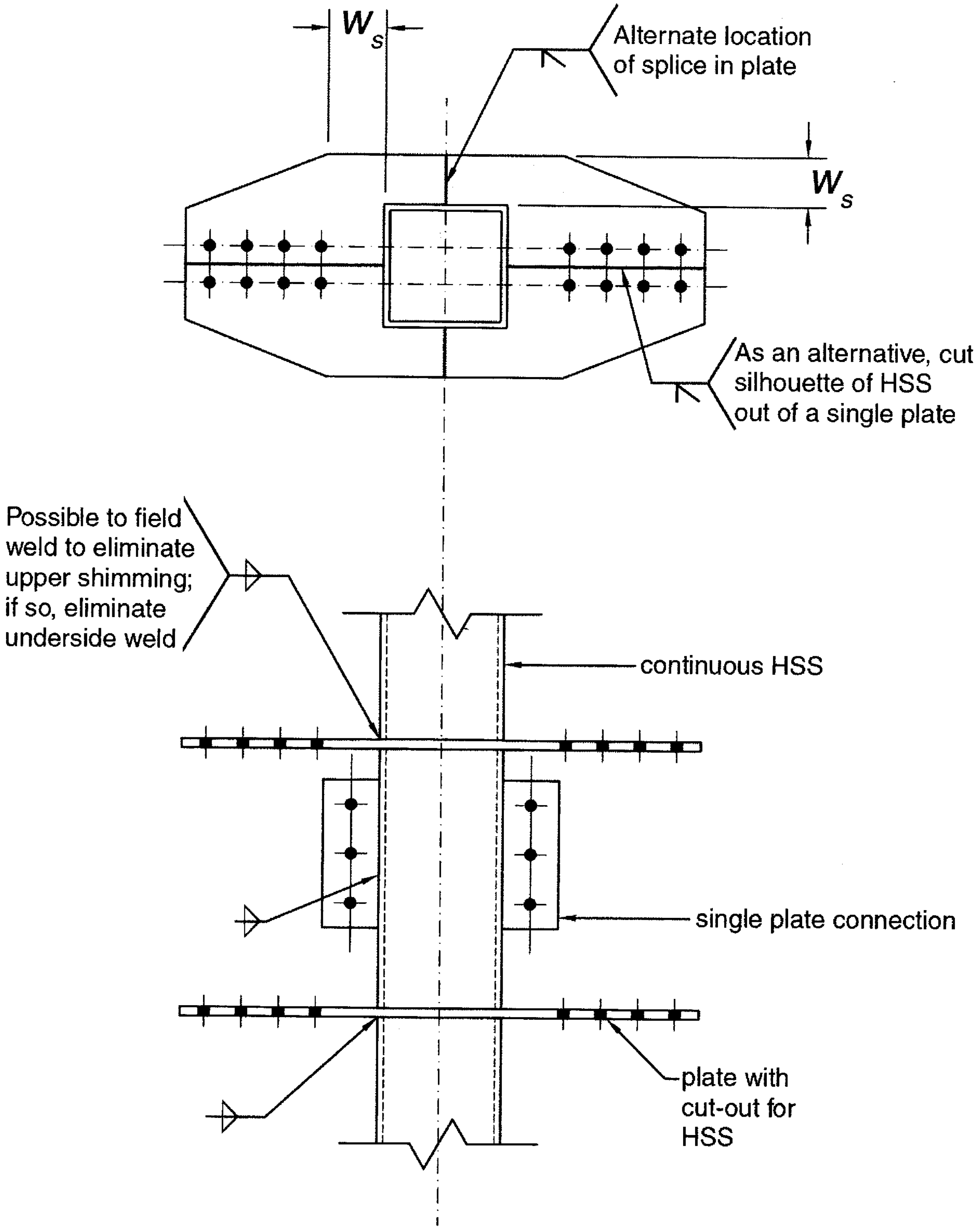


Figure 12–17. Exterior plate moment connection.

Design Considerations for HSS End-Plate FR Moment Connections

HSS end-plates can be bolted to either an end-plate welded to the face of the HSS as in Figure 12–18a or to angles welded to the sides of the HSS as is illustrated in Figure 12–18b. The projection of the end plate beyond the sides of the HSS may interfere with the construction of other building components.

For this connection to be practical, the flange width should be as large as or larger than the HSS width. It is reasonable to consider only two bolts near each flange tip as effective in tension on the beam end plate. For the end plate type connection, the buckling strength of the HSS side wall must be checked.

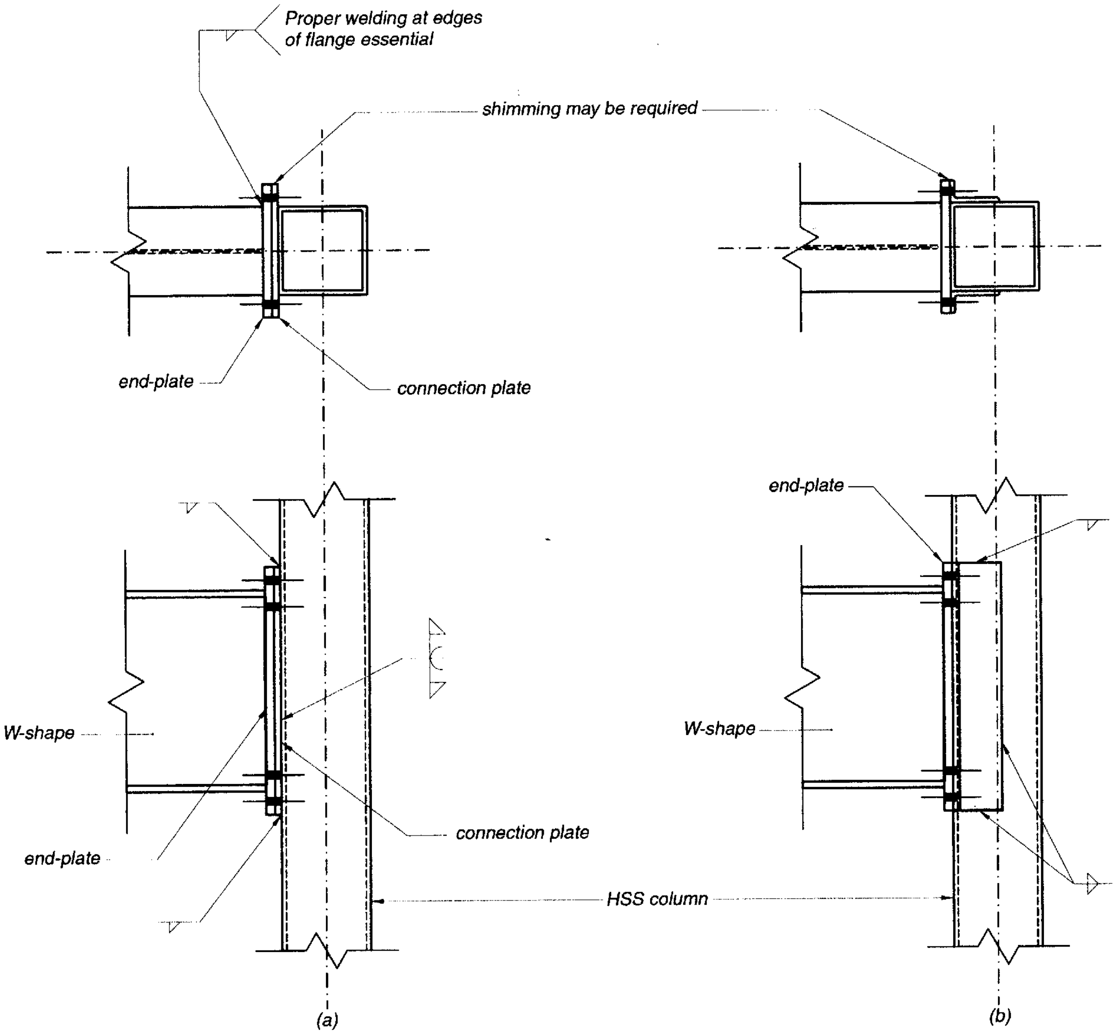


Figure 12–18. End-plate splices to HSS columns.

HSS Columns Above and Below Continuous Beams

Field connection to the flanges of the beam and of continuous beams can be used at joints where there is an HSS above and below a continuous beam. This situation is illustrated in Figures 12–19 and 12–20. If the column load is not high, stiffener plates may be used to transfer the axial load across the beam as shown in Figure 12–19a. If the axial load is higher, it may be necessary to use a split HSS instead of plate stiffeners, as shown in Figure 12–19b. The width of the W-shape must be at least as wide as the HSS and should preferably be wider than the HSS for this detail to be used as shown. It may be necessary to use a rectangular HSS column in order to fit the HSS base plate on the beam flange. The moment transfer to the HSS is limited by the strength of the four bolts, the W-shape flange thickness, and the base and cap plate thicknesses.

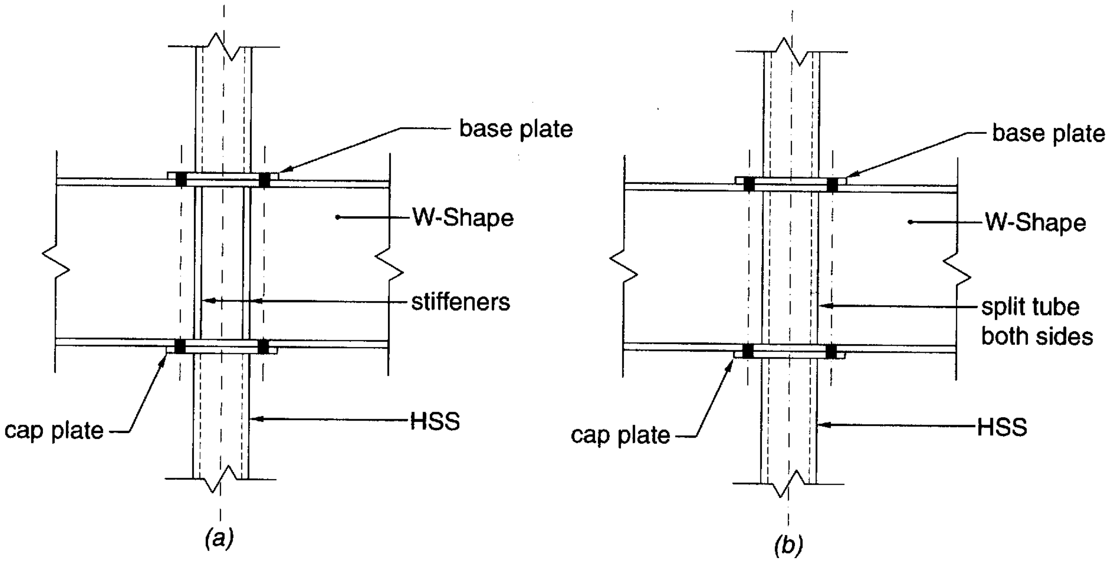


Figure 12–19. HSS columns spliced to continuous beams.

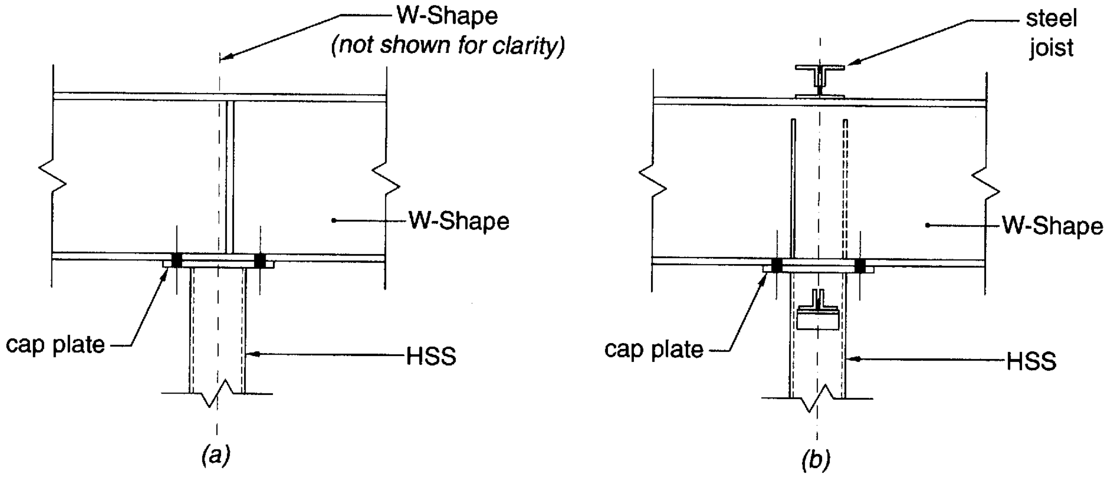
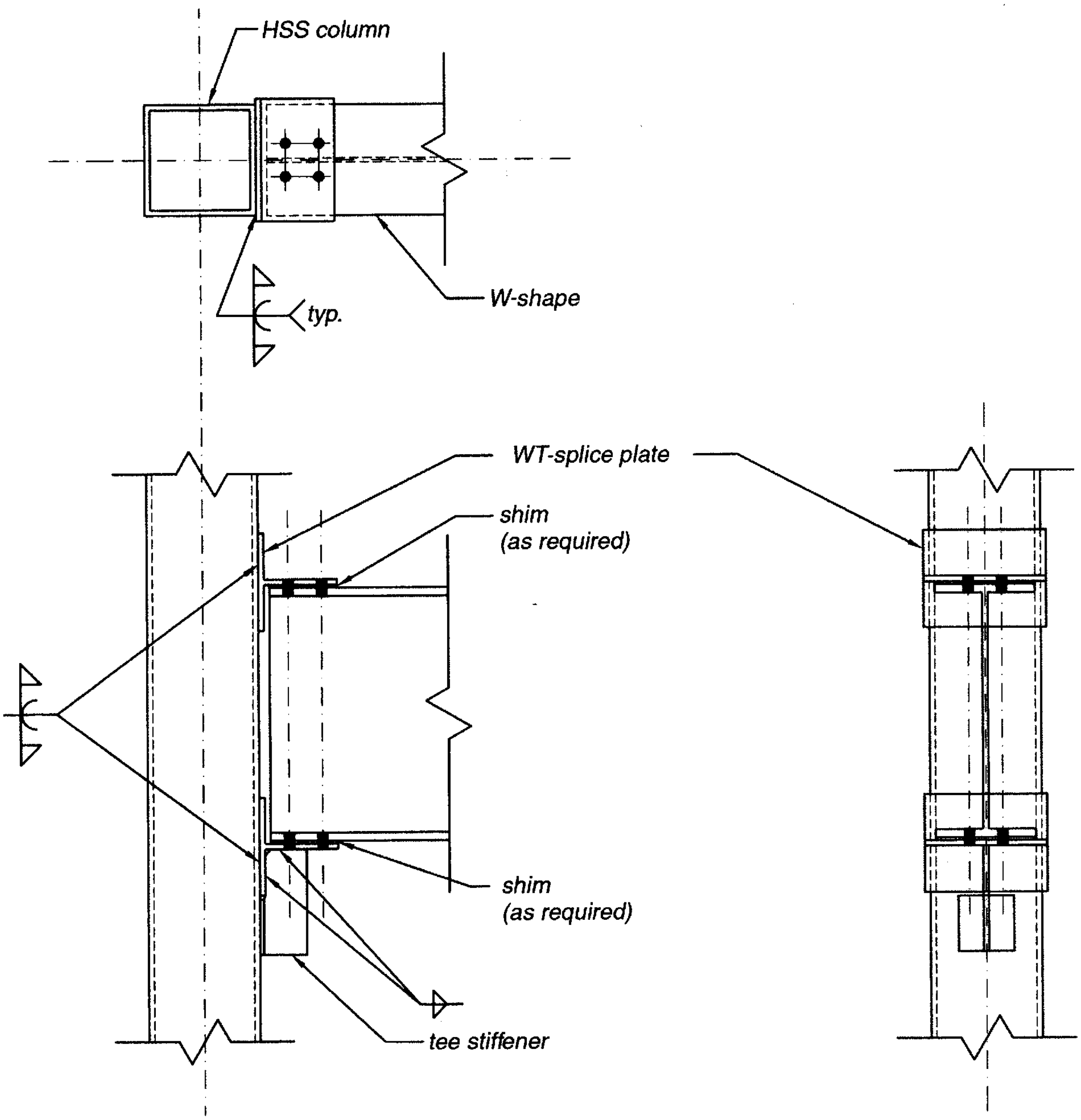


Figure 12–20. Roof beam continuous over HSS column.

HSS Welded Tee Flange Connections

If the primary moment transfer is from a wide flange to an HSS, rather than through the HSS to another wide flange, a number of other connection concepts will work well. One of these is to use structural tee sections to transfer the force from the flanges of the wide flange to the walls of the HSS as is illustrated in Figure 12–21. The tees should be long enough so that a flare bevel-groove (or single J-groove) weld with weld reinforcement can be used to connect the tee to the HSS. An alternative to using the tees to transfer the beam shear would be to use a single plate connection, if a deep enough plate can be fit between the flanges of the tees.

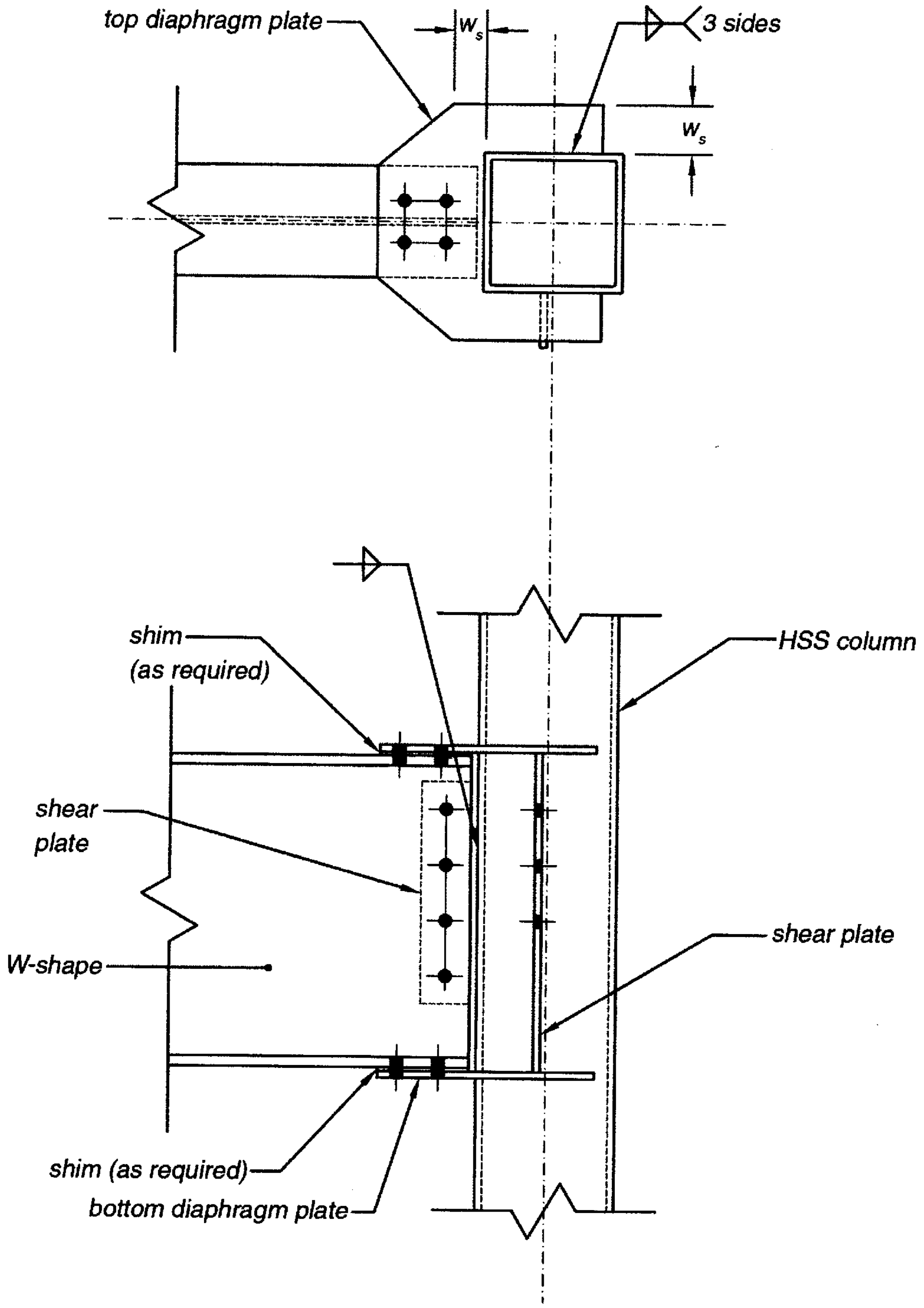


Note: A shear plate could be used in lieu of the vertical tee stiffener.

Figure 12–21. Tee splice plates to HSS column.

HSS Diaphragm Plate Connections

If the moment delivered by the W-shape to the HSS cannot be transmitted by other means, then use of diaphragm plates that transfer the flange loads to the sides of the HSS is appropriate. This is illustrated in Figure 12–22. For this moment connection the limit states are those indicated for the cut-out plate connection plus a check of the weld transferring shear from the flange plate to the HSS wall.

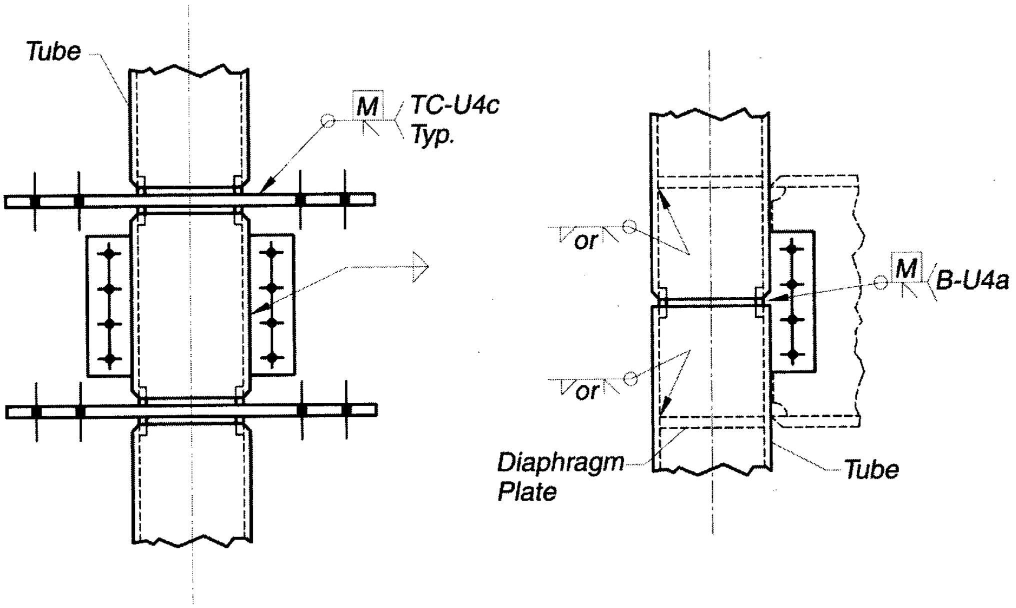


Note: A stiffened seat could also be used in lieu of the shear plate.

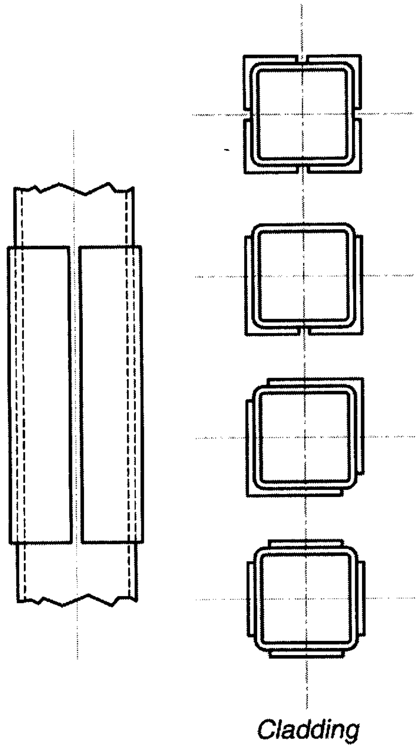
Figure 12–22. Diaphragm plate splice to exterior HSS column.

Suggested Details for HSS to Wide-Flange Moment Connections

The details shown in Figures 12–23 and 12–24 are suggested details only and are not intended to prohibit the use of other connection details.

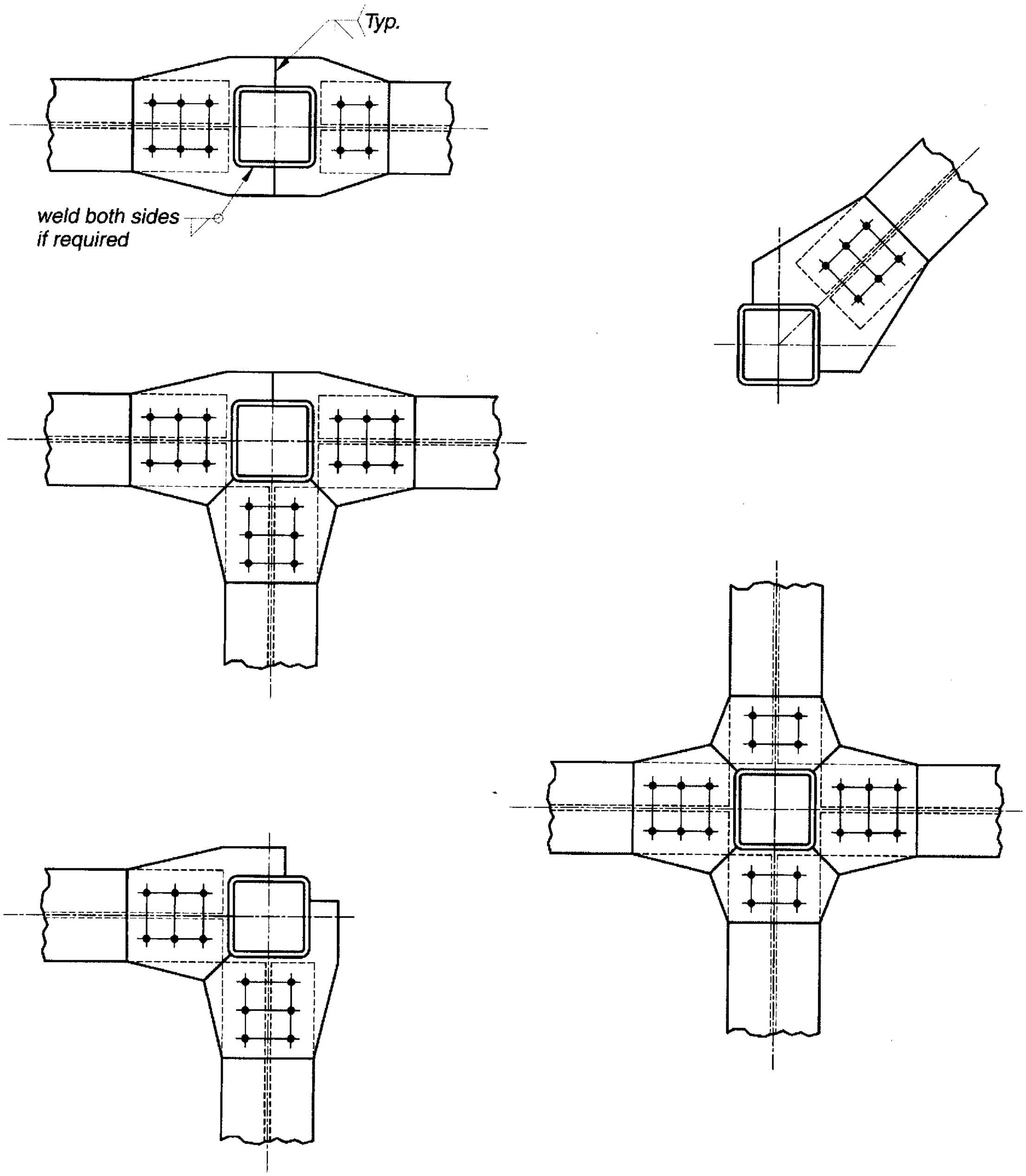


Through - Plate Diaphragm *Interior Plate Diaphragm*



HSS Column Reinforcement

Figure 12–23. Suggested detail.



Note: Shear connections not shown for clarity .

Figure 12–24. Suggested detail.

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PART 13

DESIGN OF BRACING CONNECTIONS AND TRUSS CONNECTIONS

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of concentric bracing connections and truss connections. For bracing connections and truss connections that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

BRACING CONNECTIONS

Diagonal Bracing Members

Diagonal bracing members can be rods, single angles, channels, double angles, tees, W-shapes, or HSS as required by the loads. Slender diagonal bracing members are relatively flexible and, thus, vibration and sag may be considerations. In slender tension-only bracing composed of light angles, these problems can be minimized with “draw” or pretension created by shortening the fabricated length of the diagonal brace from the theoretical length, L, between member working points. In general, the following deductions will be sufficient to accomplish the required draw: no deduction for $L \leq 10$ ft; deduct $^1/_{16}$ in. for 10 ft $< L \leq 20$ ft; deduct $^1/_8$ in. for $20 < L \leq 35$ ft; and, deduct $^3/_{16}$ in. for $L > 35$ ft. This approach is not applicable to heavier diagonal bracing members, since it is difficult to stretch these members; vibration and sag are not usually design considerations in heavier diagonal bracing members. In any diagonal bracing member, however, it is permissible to deduct an additional $^1/_{32}$ in. when necessary to avoid dimensioning to thirty-seconds of an inch.

When double-angle diagonal bracing members are separated, as at “sandwiched” end connections to gussets, intermittent connections must be provided if the unsupported length of the diagonal brace exceeds the limits specified in AISC Specification Section D4 for tension members or AISC Specification Section E6 for compression members. Note that a minimum of two stitch-fillers are required. These may be either bolted or welded stitch-fillers. Many fabricators prefer ring or rectangular bolted stitch-fillers when the angles require other punching, as at the end connections. In welded construction, a stitch-filler with protruding ends, as shown in Figure 13–1a is preferred because it is easy to fit and weld. The short stitch-filler shown in Figure 13–1b is used if a smooth appearance is desired.

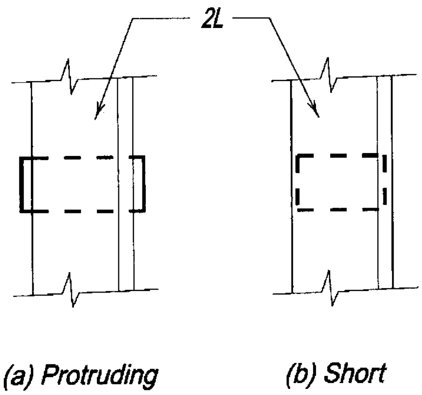


Figure 13–1. Welded stitch fillers.

When a full-length filler is provided, as in corrosive environments, the maximum spacing of stitch bolts should be as specified in AISC Specification Section J3.5. Alternatively, the edges of the filler may be seal welded.

Force Transfer in Diagonal Bracing Connections

There has been some controversy as to which of several available analysis methods provides the best means for the safe and economical design and analysis of diagonal bracing connections. To resolve this situation, starting in 1981, AISC sponsored extensive computer studies of this connection by Richard (1986). Associated with Richard's work, full-scale tests were performed by Bjorhovde and Chakrabarti (1985), Gross and Cheok (1988), and Gross (1990). Also, AISC and ASCE formed a task group to recommend a design method for this connection. In 1990, this task group recommended three methods for further study; refer to Appendix A of Thornton (1991).

Using the results of the aforementioned full scale tests, Thornton (1991) showed that these three methods yield safe designs, and that of the three methods, the Uniform Force Method (see Model 3 of Thornton, 1991) best predicts both the available strength and critical limit state of the connection. Furthermore, Thornton (1992) showed that the Uniform Force Method yields the most economical design through comparison of actual designs by the different methods and through consideration of the efficiency of force transmission. For the above reasons, and also because it is the most versatile method, the Uniform Force Method has been adopted for use in this book.

The Uniform Force Method

The essence of the Uniform Force Method is to select the geometry of the connection so that moments do not exist on the three connection interfaces; i.e., gusset-to-beam, gusset-to-column, and beam-to-column. In the absence of moment, these connections may then be designed for shear and/or tension only, hence the origin of the name Uniform Force Method.

Required Strength

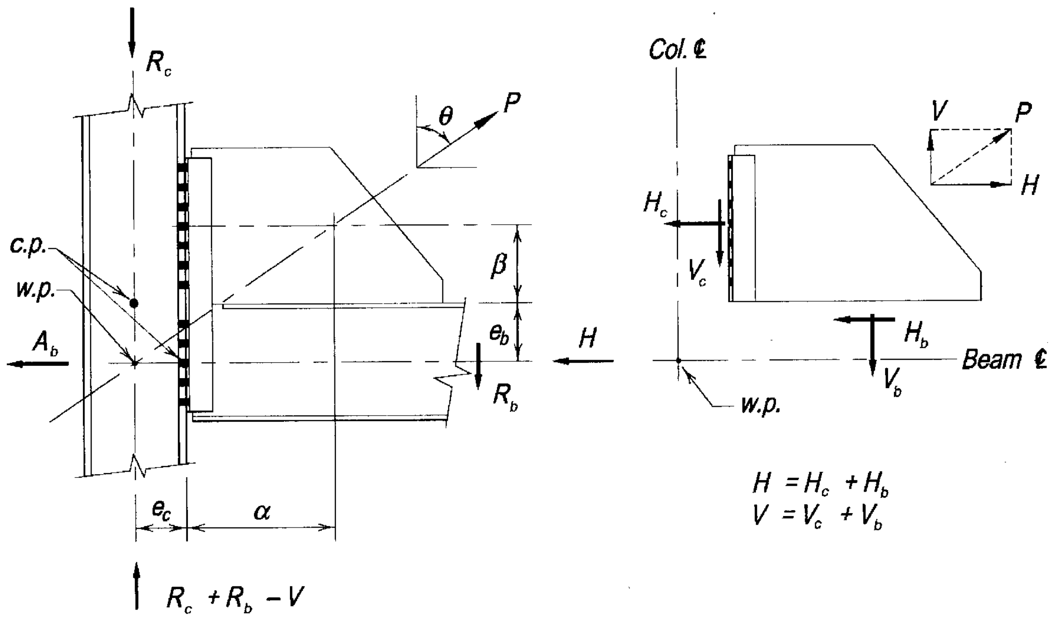
With the control points (c.p.) as illustrated in Figure 13–2 and the working point (w.p.) chosen at the intersection of the centerlines of the beam, column, and diagonal brace as shown in Figure 13–2a, four geometric parameters e_b, e_c, α, and β can be identified, where

e_b = one-half the depth of the beam, in.
e_c = one-half the depth of the column, in. Note that, for a column web support, $e_c \approx 0$.
α = distance from the face of the column flange or web to the centroid of the gusset-to-beam connection, in.
β = distance from the face of the beam flange to the centroid of the gusset-to-column connection, in.

For the force distribution shown in the free-body diagrams of Figures 13–2b, 13–2c, and 13–2d to remain free of moments on the connection interfaces, the following expression must be satisfied:

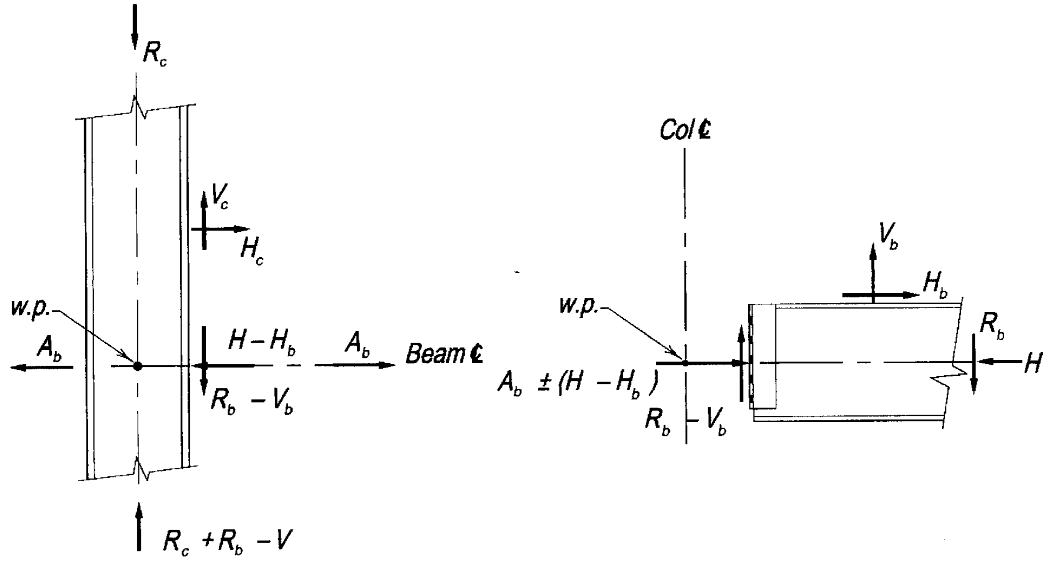
$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$$

Since the variables on the right of the equal sign (e_b, e_c, and θ) are all defined by the members being connected and the geometry of the structure, the designer may select values of α



(a) Diagonal bracing connection and external forces

(b) Gusset free-body diagram



(c) Column free-body diagram

(d) Beam free-body diagram

R = R_u or R_a, required end reaction of the beam
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the beam to gusset connection
H_c = H_{uc} or H_{ac}, required axial force on the column to gusset connection
V_b = V_{ub} or V_{ab}, required shear force on the beam to the gusset connection
V_c = V_{uc} or V_{ac}, required shear force on the column to gusset connection
P = P_u or P_a, required axial force
V = vertical component of the required force

Figure 13–2. Force transfer by the uniform force (UF) method, work point (w.p.) and control points (c.p.) as indicated.

and β for which the equation is true, thereby locating the centroids of the gusset-to-beam and gusset-to-column connections.

Once α and β have been determined, the required axial and shear forces for which these connections must be designed can be determined from the following equations:

$$V_c = \frac{\beta}{r}P \qquad H_c = \frac{e_c}{r}P$$

$$H_b = \frac{\alpha}{r}P \qquad V_b = \frac{e_b}{r}P$$

where

$$r = \sqrt{(\alpha + e_c)^2 + (\beta + e_b)^2}$$

The gusset-to-beam connection must be designed for the required shear force, H_b, and the required axial force, V_b, the gusset-to-column connection must be designed for the required shear force, V_c, and the required axial force, H_c, and the beam-to-column connection must be designed for the required shear

$$R - V_b$$

and the required axial force

$$A_b \pm (H - H_b)$$

Note that, while the axial force, P_u or P_a, is shown as a tensile force, it may also be a compressive force; were this the case the signs of the resulting gusset forces would change.

Special Case 1, Modified Working Point Location

As illustrated in Figure 13–3a, the working point in Special Case 1 of the Uniform Force Method is chosen at the corner of the gusset; this may be done to simplify layout or for a column web connection. With this assumption, the terms in the gusset force equations involving e_b and e_c drop out and the interface forces, as shown in Figures 13–3b, 13–3c, and 13–3d, are:

$$H_b = P\sin\theta = H \qquad V_b = 0$$

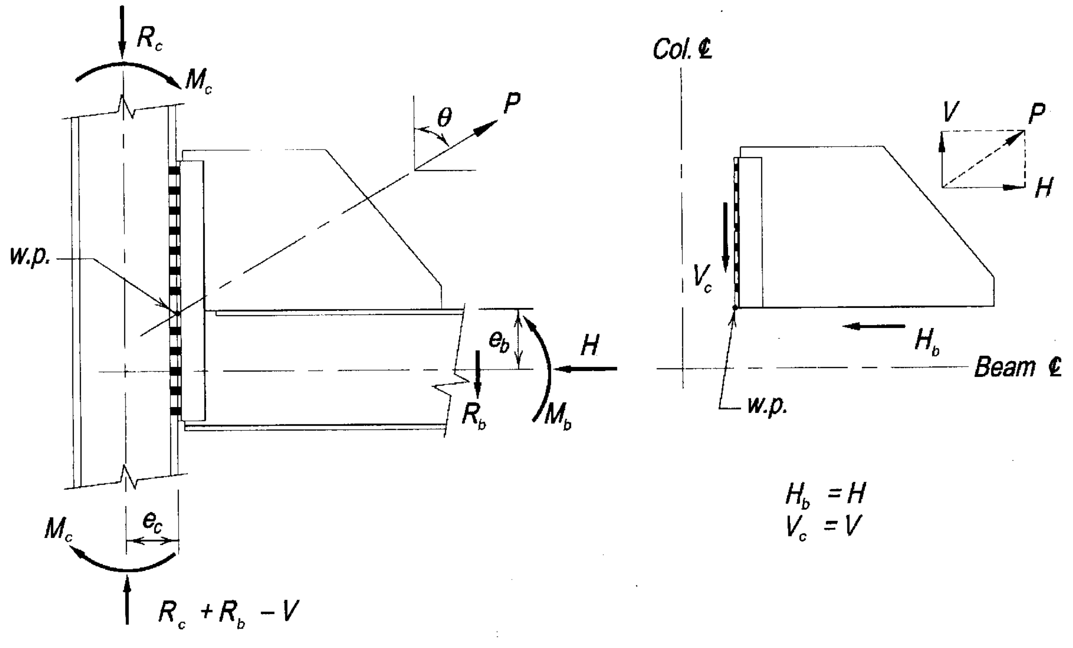
$$V_c = P\cos\theta = V \qquad H_c = 0$$

The gusset-to-beam connection must be designed for the required shear force, H_b, and the gusset-to-column connection must be designed for the required shear force, V_c. Note, however, that the change in working point requires that the beam be designed for the required moment, M_b, where

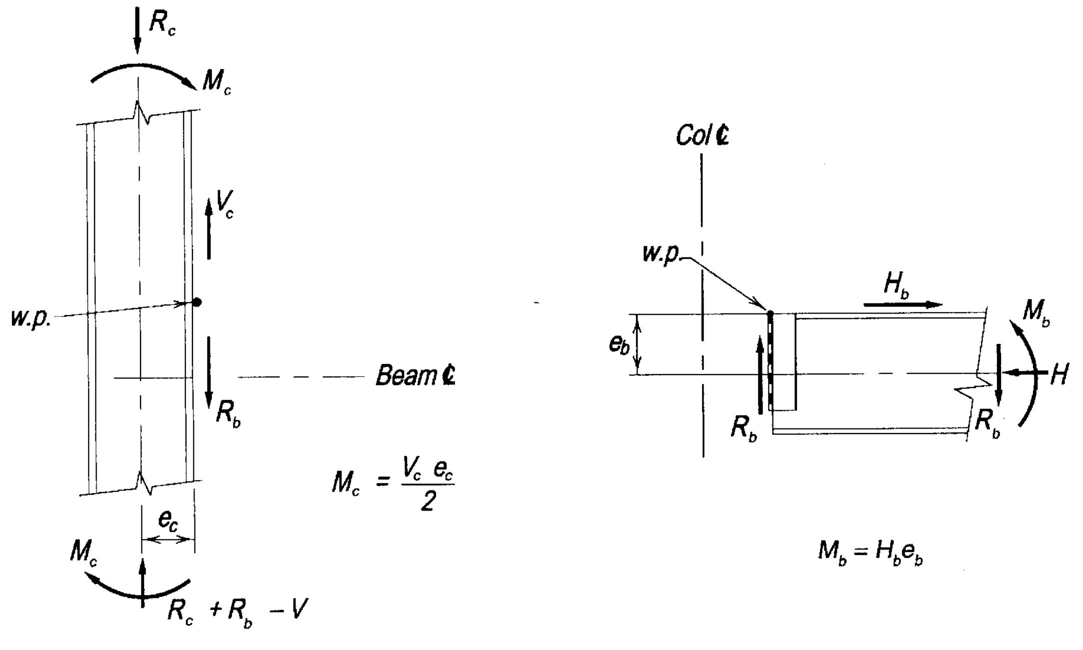
$$M_b = H_b e_b$$

and the column must be designed for the required moment, M_c. For an intermediate floor, this is determined as:

$$M_c = \frac{V_c e_c}{2}$$



(a) Diagonal bracing connection *(b) Gusset free-body diagram*



(c) Column free-body diagram *(d) Beam free-body diagram*

R = R_u or R_a, required end reaction of the beam
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the beam to gusset connection
H_c = H_{uc} or H_{ac}, required axial force on the column to gusset connection
V_b = V_{ub} or V_{ab}, required shear force on the beam to the gusset connection
V_c = V_{uc} or V_{ac}, required shear force on the column to gusset connection
P = P_u or P_a, required axial force
V = vertical component of the required force

Figure 13–3. Force transfer, UF method special case 1.

An example demonstrating this eccentric special case is presented in AISC (1984). This eccentric case was endorsed by the AISC/ASCE task group (Thornton, 1991) as a reduction of the three recommended methods when the work point is located at the gusset corner. While calculations are somewhat simplified, it should be noted that resolution of the required force P into the shears V_c and H_b may not result in the most economical connection.

Special Case 2, Minimizing Shear in the Beam-to-Column Connection

If the brace force, as illustrated in Figure 13–4a, were compressive instead of tensile and the required beam reaction, R_b, were high, the addition of the extra shear force, V_b,into the beam might exceed the available strength of the beam and require doubler plates or a haunched connection. Alternatively, the vertical force in the gusset-to-beam connection, V_b, can be limited in a manner which is somewhat analogous to using the gusset itself as a haunch.

As illustrated in Figure 13–4b, assume that V_b is reduced by an arbitrary amount, ΔV_b. By statics, the vertical force at the gusset-to-column interface will be increased to $V_c + \Delta V_b$, and a moment M_b will result on the gusset-to-beam connection, where

$$M_b = (\Delta V_b)\alpha$$

If ΔV_b is taken equal to V_b, none of the vertical component of the brace force is transmitted to the beam; the resulting procedure is that presented by AISC (1984) for concentric gravity axes, extended to connections to column flanges. This method was also recommended by the AISC/ASCE task group (Thornton, 1991)

Design by this method may be uneconomical. It is very punishing to the gusset and beam because of the moment M_b induced on the gusset-to-beam connection. This moment will require a larger connection and a thicker gusset. Additionally, the limit state of local web yielding may limit the strength of the beam. This special case interrupts the natural flow of forces assumed in the Uniform Force Method and thus is best used when the beam-to-column interface is already highly loaded, independently of the brace, by a high shear, R, in the beam-to-column connection.

Special Case 3, No Gusset-to-Column Web Connection

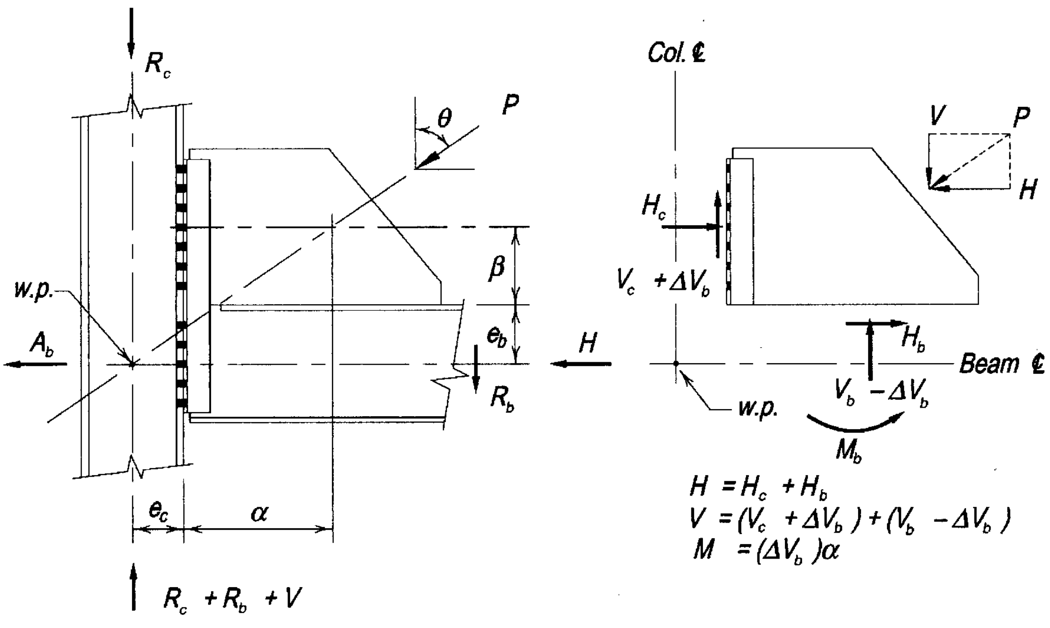
When the connection is to a column web and the brace is shallow (as for large θ) or the beam is deep, it may be more economical to eliminate the gusset-to-column connection entirely and connect the gusset to the beam only. The Uniform Force Method can be applied to this situation by setting β and e_c equal to zero as illustrated in Figure 13–5. Since there is to be no gusset-to-column connection, V_c and H_c also equal zero. Thus, $V_b = V$ and $H_b = H$.

If $\bar{\alpha} = \alpha = e_b \tan\theta$, there is no moment on the gusset-to-beam interface and the gusset-to-beam connection can be designed for the required shear force, H_b, and the required axial force, V_b. If $\bar{\alpha} \neq \alpha = e_B \tan\theta$, the gusset-to-beam interface must be designed for the moment, M_b, in addition to H_b and V_b, where

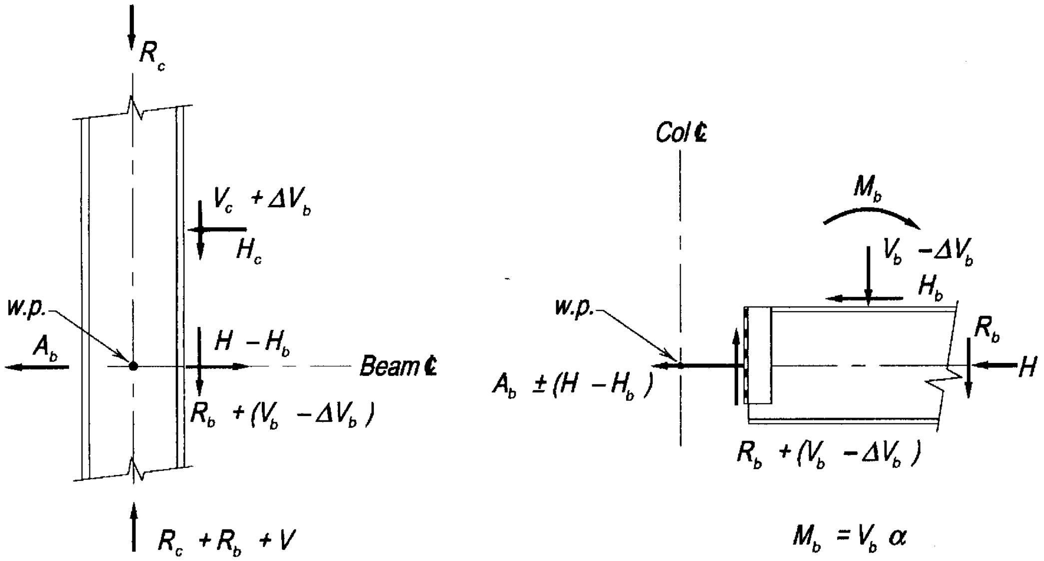
$$M_b = V_b\,(\alpha - \bar{\alpha})$$

The beam-to-column connection must be designed for the required shear force, $R + V_b$.

Note that, since the connection is to a column web, e_c is zero and hence H_c is also zero. For a connection to a column flange, if the gusset-to-column-flange connection is elimi-



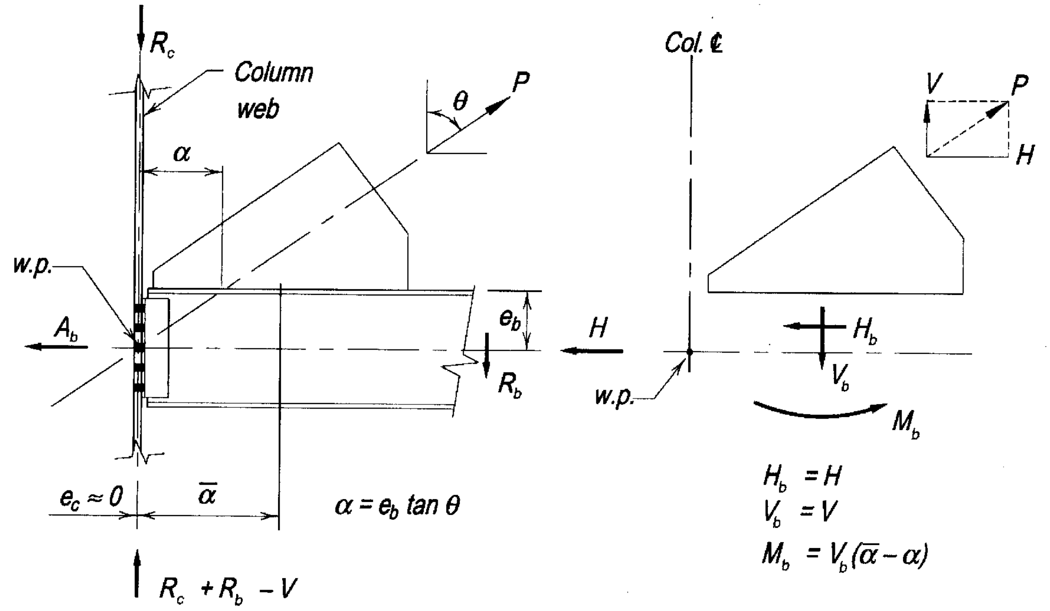
(a) Diagonal bracing connection (b) Gusset free-body diagram



(c) Column free-body diagram (d) Beam free-body diagram

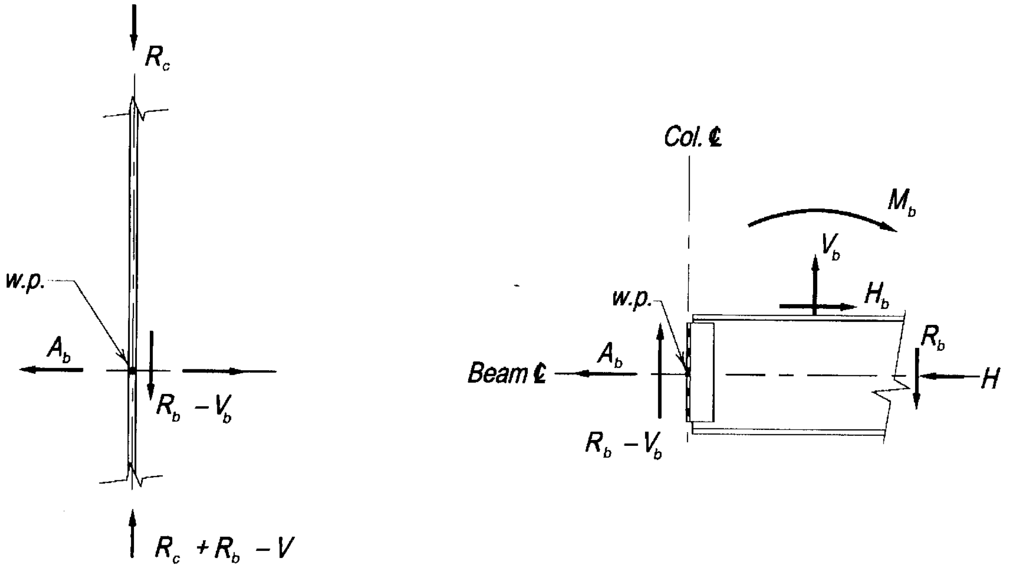
R = R_u or R_a, required end reaction of the beam
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the beam to gusset connection
H_c = H_{uc} or H_{ac}, required axial force on the column to gusset connection
V_b = V_{ub} or V_{ab}, required shear force on the beam to the gusset connection
V_c = V_{uc} or V_{ac}, required shear force on the column to gusset connection
P = P_u or P_a, required axial force
V = vertical component of the required force

Figure 13–4. Force transfer, UF method special case 2.



(a) Diagonal bracing connection

(b) Gusset free-body diagram



(c) Column free-body diagram

(d) Beam free-body diagram

R = R_u or R_a, required end reaction of the beam
A_b = A_{ub} or A_{ab}, required transverse force from adjacent bay
H = horizontal component of the required axial force
H_b = H_{ub} or H_{ab}, required shear force on the beam to gusset connection
H_c = H_{uc} or H_{ac}, required axial force on the column to gusset connection
V_b = V_{ub} or V_{ab}, required shear force on the beam to the gusset connection
V_c = V_{uc} or V_{ac}, required shear force on the column to gusset connection
P = P_u or P_a, required axial force
V = vertical component of the required force

Figure 13–5. Force transfer, UF method special case 3.

nated, the beam-to-column connection must be a moment connection designed for the moment, Ve_c, in addition to the shear, V. Thus, uniform forces on all interfaces are no longer possible.

Analysis of Existing Diagonal Bracing Connections

A combination of α and β which provides for no moments on the three interfaces can usually be achieved when a connection is being designed. However, when analyzing an existing connection or when other constraints exist on gusset dimensions, the values of α and β may not satisfy the basic relationship

$$\alpha - \beta \tan\theta = e_b \tan\theta - e_c$$

When this happens, uniform interface forces will not satisfy equilibrium and moments will exist on one or both gusset edges or at the beam-to-column interface.

To illustrate this point, consider an existing design where the actual centroids of the gusset-to-beam and gusset-to-column connections are at $\bar{\alpha}$ and $\bar{\beta}$, respectively. If the connection at one edge of the gusset is more rigid than the other, it is logical to assume that the more rigid edge takes all of the moment necessary for equilibrium. For instance, the gusset of Figure 13–2 is shown welded to the beam and bolted with double angles to the column. For this configuration, the gusset-to-beam connection will be much more rigid than the gusset-to-column connection.

Take α and β as the ideal centroids of the gusset-to-beam and gusset-to-column connections, respectively. Setting $\beta = \bar{\beta}$, the α required for no moment on the gusset-to-beam connection may be calculated as

$$\alpha = K + \bar{\beta}\tan\theta$$

where

$$K = e_b \tan\theta - e_c$$

If $\alpha \neq \bar{\alpha}$, a moment M_b will exist on the gusset-to-beam connection where,

$$M_b = V_b\,(\alpha - \bar{\alpha})$$

Conversely, suppose the gusset-to-column connection were judged to be more rigid. Setting $\alpha = \bar{\alpha}$, the β required for no moment on the gusset-to-column connection may be calculated as

$$\beta = \frac{\bar{\alpha} - K}{\tan\theta}$$

If $\beta \neq \bar{\beta}$, a moment, M_c, will exist on the gusset-to-column connection where,

$$M_c = H_c(\beta - \bar{\beta})$$

If both connections were equally rigid and no obvious allocation of moment could be made, the moment could be distributed based on minimized eccentricities $\alpha - \bar{\alpha}$ and $\beta - \bar{\beta}$ by minimizing the objective function, ξ, where

$$\xi = \left(\frac{\alpha - \bar{\alpha}}{\bar{\alpha}}\right)^2 + \left(\frac{\beta - \bar{\beta}}{\bar{\beta}}\right)^2 - \lambda\left(\alpha - \beta\tan\theta - K\right)$$

In the preceding equation, λ is a Lagrange multiplier.

The values of α and β that minimize ξ are

$$\alpha = \frac{K' \tan\theta + K\left(\dfrac{\bar{\alpha}}{\bar{\beta}}\right)^2}{D}$$

and

$$\beta = \frac{K' - K\tan\theta}{D}$$

where

$$K' = \bar{\alpha}\left(\tan\theta + \frac{\bar{\alpha}}{\bar{\beta}}\right)$$

$$D = \tan^2\theta + \left(\frac{\bar{\alpha}}{\bar{\beta}}\right)^2$$

Available Strength

The available strength of a diagonal bracing connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must equal or exceed the required strength, R_u or R_a. Note that when the gusset is directly welded to the beam or column, the connection should be designed for the larger of the peak stress and 1.25 times the average stress, but the weld size need not be larger than that required to develop the strength of the gusset. This 25 percent increase is recommended to provide ductility to allow adequate force redistribution in the weld group (Hewitt and Thornton, 2004).

TRUSS CONNECTIONS

Members in Trusses

For light loads, trusses are commonly composed of tees for the top and bottom chords with single-angle or double-angle web members. In welded construction, the single-angle and double-angle web members may, in many cases, be welded to the stem of the tee, thus, eliminating the need for gussets. When single-angle web members are used, all web members should be placed on the same side of the chord; staggering the web members causes a torque on the chord, as illustrated in Figure 13–6.

Double-angle truss members are usually designed to act as a unit. When unequal-leg angles are used, long legs are normally assembled back to back. A simple notation for the angle assembly is LLBB (long legs back-to-back) and SLBB (short legs back-to-back). Alternatively, the notation might be graphical in nature as  and . For large loads, W-shapes may be used with the web vertical and gussets welded to the flange for the truss connections. Web members may be single angles or double angles, although W-shapes are sometimes used for both chord and web members as shown in Figure 13–7. Heavy shapes in trusses must meet the design and fabrication restrictions and special requirements in AISC Specification Sections A3.1c and A3.1d. With member orientation as shown for the field-welded truss joint in Figure 13–7a, connections usually are made by groove welding flanges to flanges and fillet welding webs directly or indirectly by the use of gussets. Fit-up of joints in this type of construction are very sensitive to dimensional variations in the rolled shapes; fabricators sometimes prefer to use built-up shapes in these cases.

The web connection plate in Figure 13–7a is a typical detail. While the diagonal member could theoretically be cut so that the diagonal web would be extended into the web of the chord for a direct connection, such a detail is difficult to fabricate. Additionally, welding access becomes very limited; note the obvious difficulty of welding the gusset or diagonal directly to the chord web. As illustrated, this weld is usually omitted.

When stiffeners and doubler plates are required for concentrated flange forces, the designer should consider selecting a heavier section to eliminate the need for stiffening. Although this will increase the material cost of the member, the heavier section will likely provide a more economical solution due to the reduction in labor cost associated with the elimination of stiffening (Ricker, 1992 and Thornton, 1992).

Minimum Connection Strength

Truss connections are recommended to be designed for a minimum required strength of 10 kips for LRFD or 6 kips for ASD, as noted in AISC Specification Commentary Section J1.1. Additionally, when trusses are shop assembled or field assembled on the ground for subsequent erection, consideration should be given to loads induced during handling, shipping, and erection.

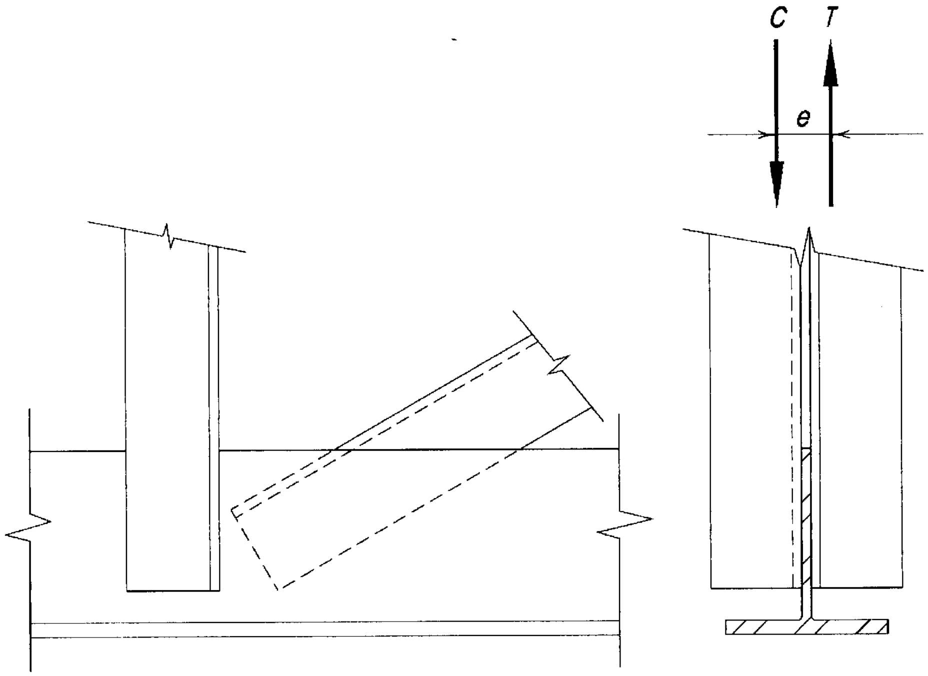


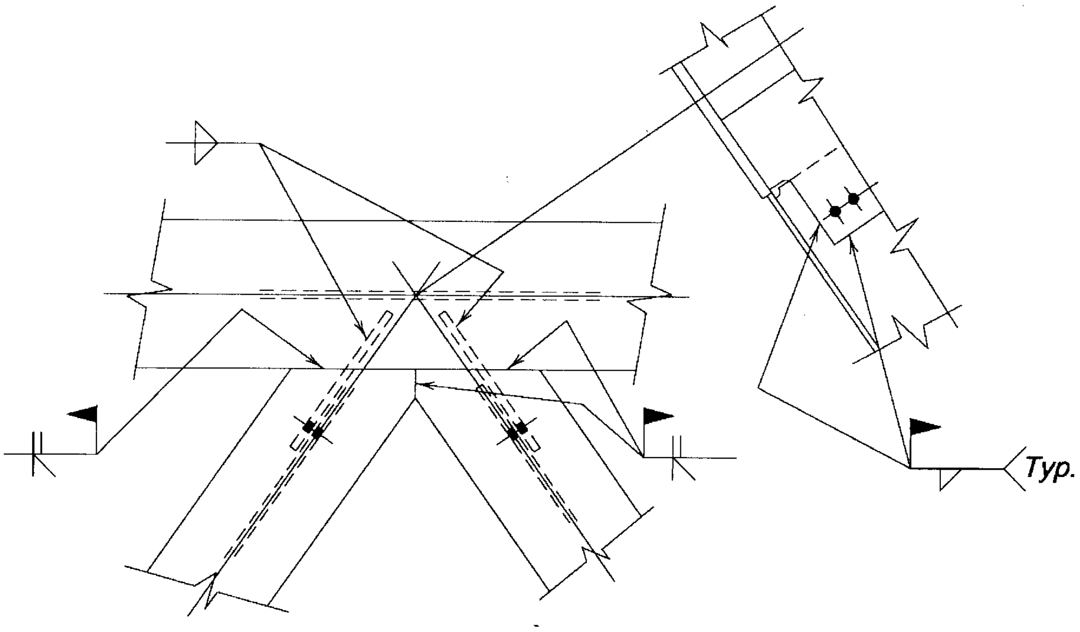
Figure 13–6. Staggered web members result in a torque on the truss chord.

Panel-Point Connections

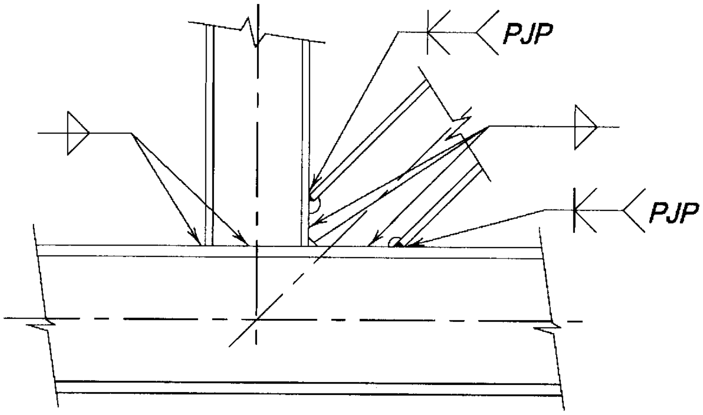
A panel-point connection connects diagonal and/or vertical web members to the chord member of a truss. These web members deliver axial forces, tensile or compressive, to the truss chord. In bolted construction, a gusset is usually required because of bolt spacing and edge distance requirements. In welded construction, it is sometimes possible to eliminate the need for a gusset.

Design Checks

The available strength of a panel-point connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connecting elements (see



(a) Shop and field welding



(b) Shop welding

Note: Check vertical and chord for reinforcing requirements

Figure 13–7. Truss panel-point connections for W-shape truss members.

Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

In the panel-point connection of Figure 13–8, the neutral axes of the vertical and diagonal truss members intersect on the neutral axis of the truss chord. As a result, the forces in all members of the truss are axial. It is common practice, however, to modify working lines slightly from the gravity axes to establish repetitive panels and avoid fractional dimensions less than $^1/_8$ in. or to accommodate a larger panel-point connection or a connection for bottom-chord lateral bracing, a purlin, or a sway-frame. This eccentricity and the resulting moment must be considered in the design of the truss chord.

In contrast, for the design of the truss web members, AISC Specification Section J1.7 permits that the center of gravity of the end connection of a statically loaded truss member need not coincide with the gravity axis of the connected member. This is because tests have shown that there is no appreciable difference in the available strength between balanced and unbalanced connections subjected to static loading. Accordingly, the truss web members and their end connections may be designed for the axial load, neglecting the effect of this minor eccentricity.

Shop and Field Practices

In bolted construction, it is convenient to use standard gage lines of the angles as truss working lines; where wider angles with two gage lines are used, the gage line nearest the heel of the angle is the one which is substituted for the gravity axis.

To provide for stiffness in the finished truss, the web members of the truss are extended to near the edge of the fillet of the tee (*k*-distance). If welded, the required welds are then applied along the heel and toe of each angle, beginning at their ends rather than at the edge of the tee stem.

Support Connections

A truss support connection connects the ends of trusses to supporting members.

Design Checks

The available strength of a support connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8) and connecting elements (see Part 9).

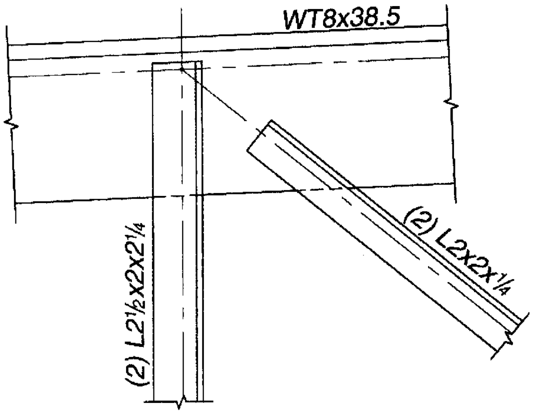


Figure 13–8. Truss panel-point connection.

Additionally, truss support connections produce tensile or compressive single concentrated forces at the beam end; the limit states of the available flange strength in local bending and the limit states of the available web strength in local yielding, crippling, and compression buckling may have to be checked. In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

At the end of a truss supported by a column, all member axes may not intersect at a common point. When this is the case, an eccentricity results. Typically, it is the neutral axis of the column that does not meet at the working point.

If trusses with similar reactions line up on opposite sides of the column, consideration of eccentricity would not be required since any moment would be transferred through the column and into the other truss. However, if there is little or no load on the opposite side of the column, the resulting eccentricity must be considered.

In Figure 13–9, the truss chord and diagonal intersect at a common working point on the face of the column flange. In this detail, there is no eccentricity in the gusset, gusset-to-column connection, truss chord, or diagonal. However, the column must be designed for the moment due to the eccentricity of the truss reaction from the neutral axis of the column.

For the truss support connection illustrated in Figure 13–10, this eccentricity results in a moment. Assuming the connection between the members is adequate, joint rotation is resisted by the combined flexural strength of the column, the truss top chord, and the truss diagonal. However, the distribution of moment between these members will be proportional to the stiffness of the members. Thus, when the stiffness of the column is much greater than the stiffness of the other elements of the truss support connection, it is good practice to design the column and gusset-to-column connection for the full eccentricity.

Due to its importance, the truss support connection is frequently shown in detail on the design drawing.

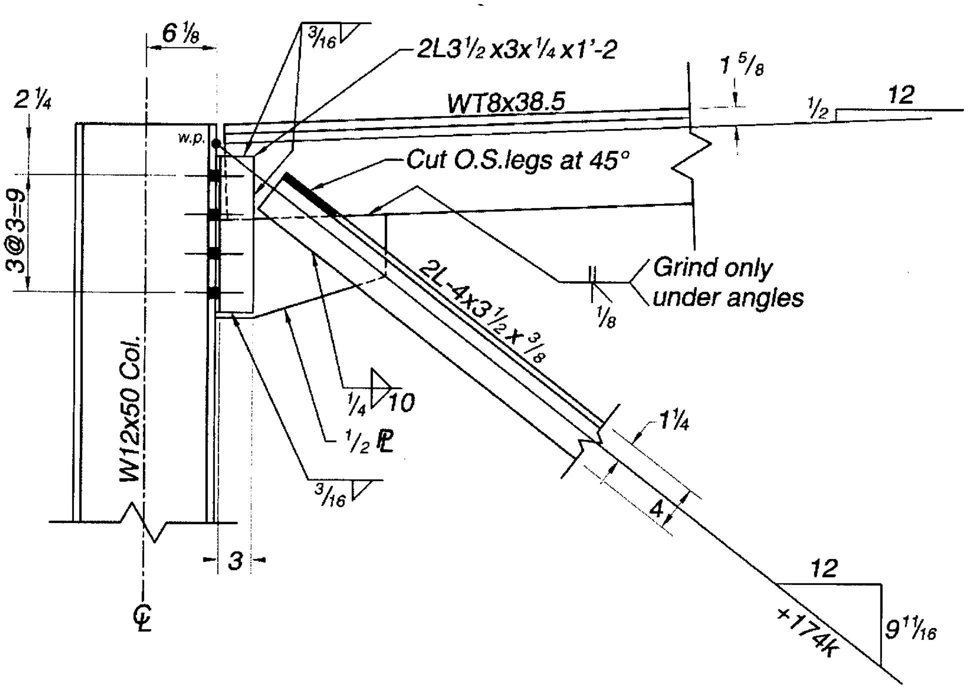


Figure 13–9. Truss support connection, working point (w.p.) on column face.

Shop and Field Practices

When a truss is erected in place and loaded, truss members in tension will lengthen and truss members in compression will shorten. At the support connection, this may cause the tension chord of a "square-ended" truss to encroach on its connection to the supporting column. When the connection is shop-attached to the truss, erection clearance must be provided with shims to fill out whatever space remains after the truss is erected and loaded. In field erected connections, however, provision must be made for the necessary adjustment in the connection.

When the tension chord delivers no calculated force to the connection, adjustment can usually be provided with slotted holes. For short spans with relatively light loads, the comparatively small deflections can be absorbed by the normal hole clearances provided for bolted construction. Slightly greater misalignment can be corrected in the field by reaming the holes. If appreciable deflection is expected, the connection may be welded. Alternatively, bolt holes may be field-drilled, but this is an expensive operation which should be avoided if at all possible.

An approximation of the elongation, Δ, can be determined as

$$\Delta = \frac{Pl}{AE}$$

where

Δ = elongation in inches
P = axial force due to service loads, kips
A = gross area of the truss chord, in.^{2}
l = length, in.

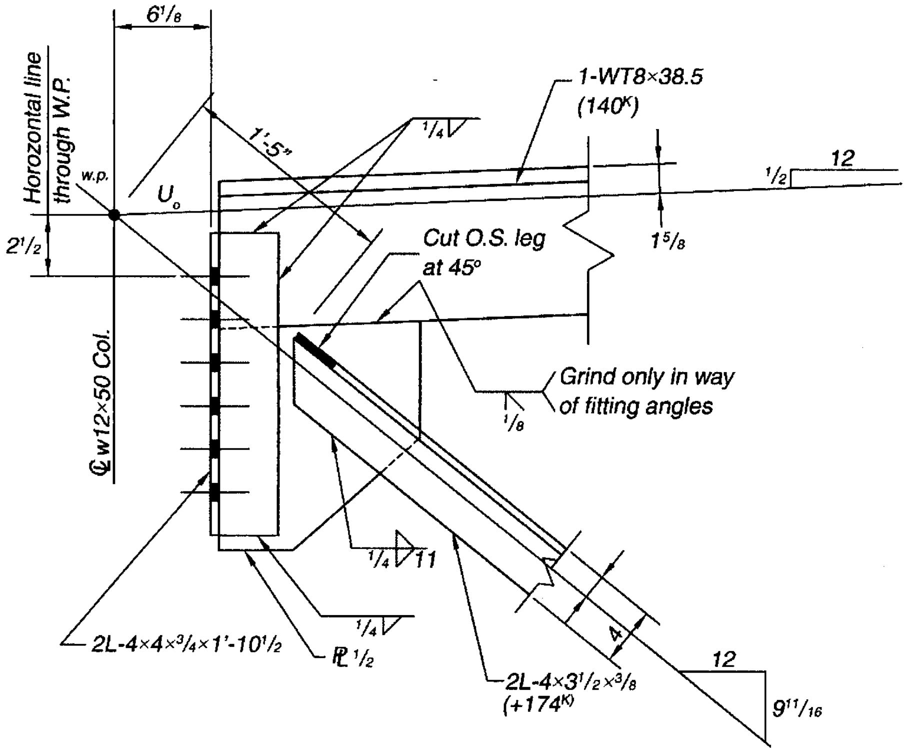


Figure 13–10. Truss-support connection, working point (w.p.) at column centerline.

The total change in length of the truss chord is $\Sigma\Delta_i$, the sum of the changes in the lengths of the individual panel segments of the truss chord. The misalignment at each support connection of the tension chord is one-half the total elongation.

Truss Chord Splices

Truss chord splices are expensive to fabricate and should be avoided whenever possible. In general, chord splices in ordinary building trusses are confined to cases where:

1. the finished truss is too large to be shipped in one piece;
2. the truss chord exceeds the available material length;
3. the reduction in member size of the chord justifies the added cost of a splice; or
4. a sharp change in direction occurs in the working line of the chord and bending does not provide a satisfactory alternative.

Splices at truss chord ends that are finished to bear should be designed in accordance with AISC Specification Section J1.4b.

Design Considerations for HSS to HSS Truss Connections

For the design of HSS-to-HSS truss connections, see AISC Specification Section K2.

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PART 14

DESIGN OF BEAM BEARING PLATES, COLUMN BASE PLATES, ANCHOR RODS, AND COLUMN SPLICES

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SCOPE

The specification requirements and other design considerations summarized in this Part apply to the design of beam bearing plates, column base plates, anchor rods and column splices. For complete coverage of column base plate connections, see AISC Design Guide 1, *Column Base Plates* (DeWolf, 1990). For bearing plates, column base plates, anchor rods and column splices that are part of a seismic force resisting system in which the seismic response modification factor, R, is taken greater than 3, the requirements in the AISC *Seismic Provisions for Structural Steel Buildings* also apply. The AISC *Seismic Provisions for Structural Steel Buildings* is available in Part 6 of the AISC *Seismic Design Manual* from the American Institute of Steel Construction, Inc. at www.aisc.org.

BEAM BEARING PLATES

A beam bearing plate is made with a plate as illustrated in Figure 14–1.

Force Transfer

The required strength (beam end reaction), R_u or R_a, is distributed from the beam bottom flange to the bearing plate over an area equal to $N \times 2k$, where N is the bearing length (length of contact between the beam bottom flange and the bearing plate), in. The bearing plate is then assumed to distribute the beam end reaction to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the plate. The bearing plate cantilever dimension is taken as

$$n = \frac{B}{2} - k$$

where B is the bearing plate width, in.

In the rare case where a bearing plate is not required, the beam end reaction, R_u or R_a, is assumed to be uniformly distributed from the beam bottom flange to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the beam flanges. The beam-flange cantilever dimension is calculated as for a bearing plate, but using the beam flange width, b_f, in place of B.

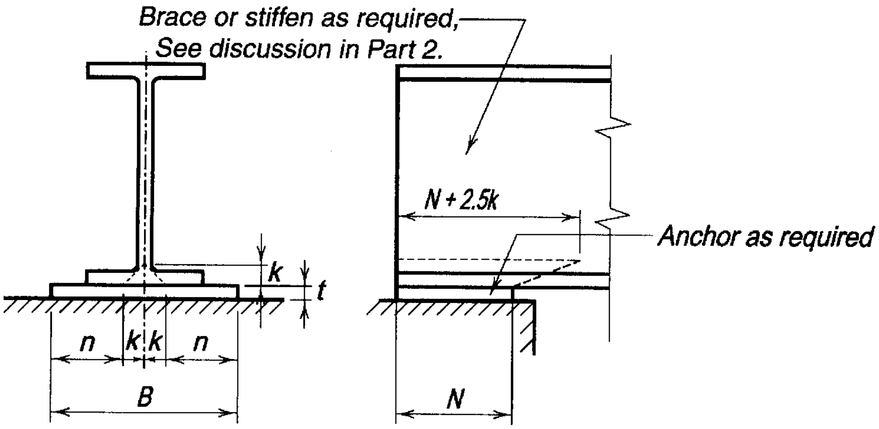


Figure 14–1. Beam bearing plate variables.

Recommended Bearing Plate Dimensions and Thickness

The length of bearing, N, may be established by available wall thickness, clearance requirements, or by the minimum requirements based on local web yielding or web crippling. The selected dimensions of the bearing plate, B and N, should preferably be in full in. Bearing plate thickness should be specified in multiples of $^1/_8$ in. up to $1^1/_4$-in. thickness and in multiples of $^1/_4$ in. thereafter.

Available Strength

The available strength of a beam bearing plate is determined from the applicable limit states from Part 9 (connection elements). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a. The stability of the beam end must also be addressed as discussed in "Design Basis, Stability Bracing, Beam Ends Supported on Bearing Plates" in Part 2.

COLUMN BASE PLATES FOR AXIAL COMPRESSION

A column base plate is made with a plate and a minimum of four anchor rods as illustrated in Figure 14–2. The base plate is often attached to the bottom of column in the shop.

Force Transfer

In Figure 14–3, the required strength (column axial force), P_u or P_a, is distributed from the column end to the column base plate in direct bearing. The column base plate is then assumed to distribute the column axial force to the concrete or masonry as a uniform bearing pressure by cantilevered bending of the plate. The critical base plate cantilever dimension, l, is determined as the larger of m, n, and $\lambda n'$ where

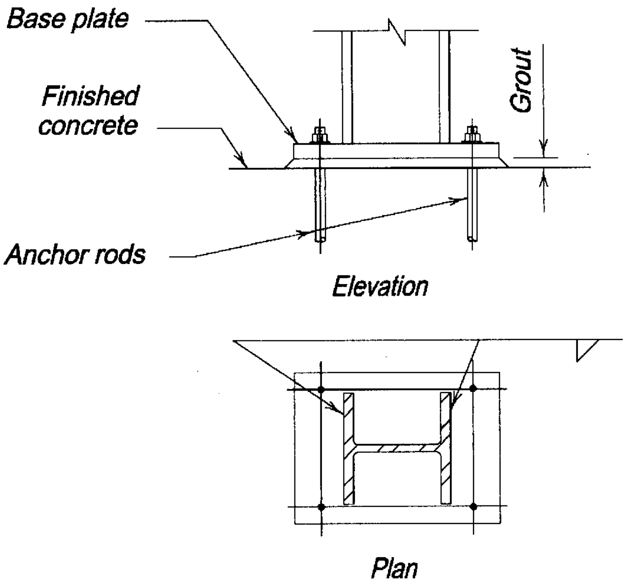


Figure 14–2. Typical column base for axial compressive loads.

$$m = \frac{N - 0.95d}{2}$$

$$n = \frac{B - 0.8b_f}{2}$$

$$n' = \frac{\sqrt{db_f}}{4}$$

$$\lambda = \frac{2\sqrt{X}}{1 + \sqrt{1 - X}} \le 1$$

LRFD	ASD
$X = \left(\frac{4db_f}{(d + b_f)^2}\right)\frac{P_u}{\phi_c P_p}$	$X = \left(\frac{4db_f}{(d + b_f)^2}\right)\frac{\Omega_c P_a}{P_p}$

Note that, because both the term in parentheses and the ratio of the required strength, P_u *or* P_a, to the available strength, $\phi_c P_p$ or P_n/Ω, are always less than or equal to 1, the value of X will always be less than or equal to 1. Note also that λ can always be taken conservatively as 1. For further information, see Thornton (1990) and AISC Design Guide No. 1 *Column Base Plates* (DeWolf and Ricker, 1990).

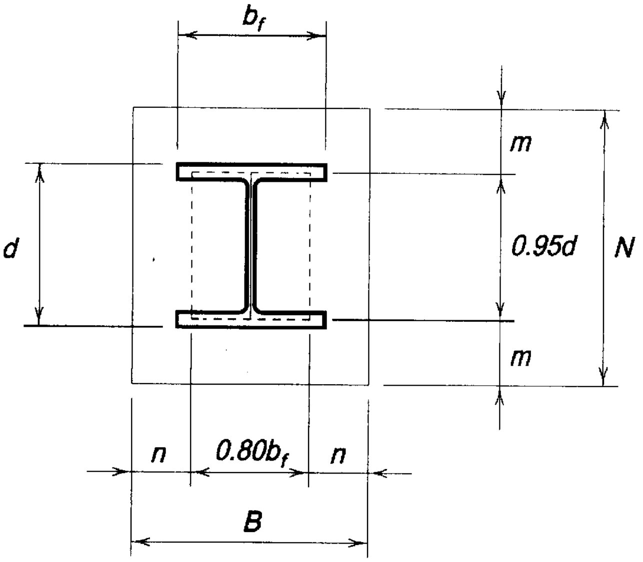


Figure 14–3. Column base-plate design variables.

Recommended Base plate Dimensions and Thickness

The selected dimensions of the base plate B and N should preferably be in full in. Base plate thickness should be specified in multiples of 1/8 in. up to 1 1/4 in. thickness and in multiples of 1/4 in. thereafter.

Available Strength

The available strength of an axially loaded column base plate is determined from the applicable limit states in Part 9 (connection elements). From Thornton (1990), the minimum base plate thickness can be calculated as

LRFD	ASD
$t_{min} = l\sqrt{\dfrac{2P_u}{0.9F_yBN}}$	$t_{min} = l\sqrt{\dfrac{3.33P_a}{F_yBN}}$

In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.

Finishing Requirements

Base plate finishing requirements are given in AISC Specification Section M2.8. When finishing is required, the plate material must be ordered thicker than the specified base plate thickness to allow for the material removed in finishing. Finishing allowances are given in Table 14–1 per ASTM A6 flatness tolerances for steel base plates with F_u equal to or less than 60 ksi based upon the width, thickness, and whether one or both sides are to be finished. Finishing allowances for steel base plates with F_u greater than 60 ksi should be increased by 50 percent.

The criteria for fit-up of column splices given in AISC Specification Section M4.4 are also applicable to column base plates.

Holes for Anchor Rods and Grouting

Recommended maximum anchor rod hole sizes are given in Table 14–2. These hole sizes will accommodate reasonable misalignments in the setting of the anchor rods and allow better adjustment of the column base to the correct centerlines. It is normally unnecessary to deduct the area of holes when determining the required base plate area. An adequate washer should be provided for each anchor rod.

When base plates with large areas are used, at least one grout hole should be provided near the center of the base plate through which grout may be placed. This will provide for a more even distribution of the grout and also prevent air pockets. Note that a grout hole may not be required when the grout is dry-packed. Grout holes do not require the same accuracy for size and location as anchor rod holes.

Holes in base plates for anchor rods and grouting often must be flame-cut, because drill sizes and punching capabilities may be limited to smaller diameters. Flame-cut holes may have a slight taper and should be inspected to assure proper clearances for anchor rods.

Grouting and Leveling

High-strength, non-shrink grout is placed between the column base plate and the supporting foundation. When base plates are shipped attached to the column, three methods of column support are:

1. The use of leveling nuts and, in some case, washers on the anchor rods beneath the base plate, as illustrated in Figure 14–4.
2. The use of shim stacks between the base plate and the supporting foundation.
3. The use of a steel leveling plate (normally 1/4 in. thick), set to elevation and grouted prior to the setting of the column. The leveling plate should meet the flatness tolerances specified in ASTM A6. It may be larger than the base plate to accommodate anchor rod placement tolerances and can be used as a setting template for the anchor rods.

For further information on grouting and leveling of column base plates, see AISC Design Guide No. 10 *Erection Bracing of Low-Rise Structural Steel Frames* (Fisher and West, 1997).

When base plates are shipped loose, the base plates are usually grouted after the base plate has been aligned and leveled with one of the preceding methods. For heavy loose base plates, three-point leveling bolts, illustrated in Figure 14–5, are commonly used. These threaded attachments may consist of a nut or an angle and nut welded to the base plate. Leveling bolts must be of sufficient length to compensate for the space provided for grouting. Rounding the point of the leveling bolt will prevent it from "walking" or moving laterally as it is turned. Additionally, a small steel pad under the point reduces friction and prevents damage to the concrete.

Heavy loose base plates should be provided with some means of handling at the erection site. Lifting holes can be provided in the vertical legs of shop-attached connection angles. Lifting lugs can also be used and can remain in place after erection, unless they create an interference or removal is required in the contract documents.

Leveling bolts or nuts should not be used to support the column during erection. If grouting is delayed until after steel erection, the base plate must be shimmed to properly distribute loads to the foundation without overstressing either the base plate or the concrete. This difficulty of supporting columns while leveling and grouting their bases makes it advisable that

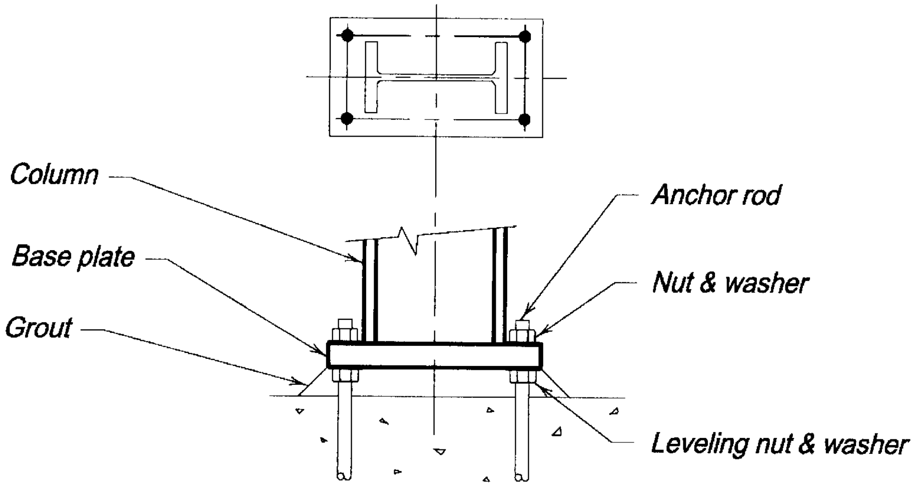


Figure 14–4. Leveling nuts and washers.

footings be finished to near the proper elevation (Ricker, 1989). The top of the rough footing should be set approximately 1 to 2 in. below the bottom of the base plate to provide for adjustment. Alternatively, an angle frame as illustrated in Figure 14–6 could be constructed to the proper elevation and filled with grout prior to erection.

COLUMN BASE PLATES FOR AXIAL TENSION, SHEAR, OR MOMENT

For anchor rod diameters not greater than 1¼ in., angles bolted or welded to the column as shown in Figure 14–7a are generally adequate to transfer uplift forces resulting from axial loads and moments. When greater resistance is required, stiffeners may be used with hori-

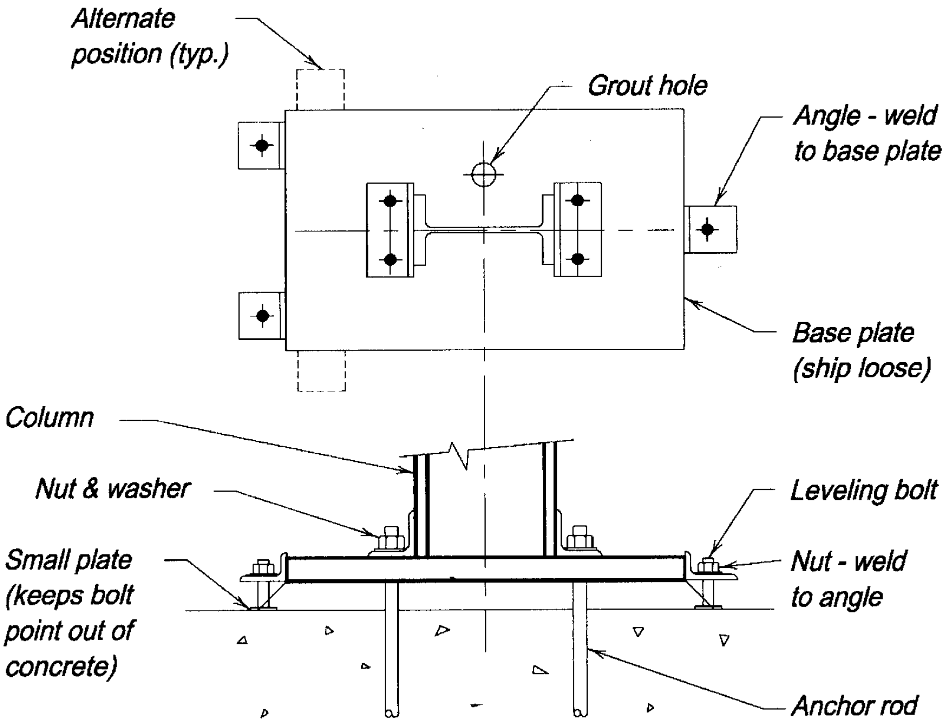


Figure 14–5. Three-point leveling.

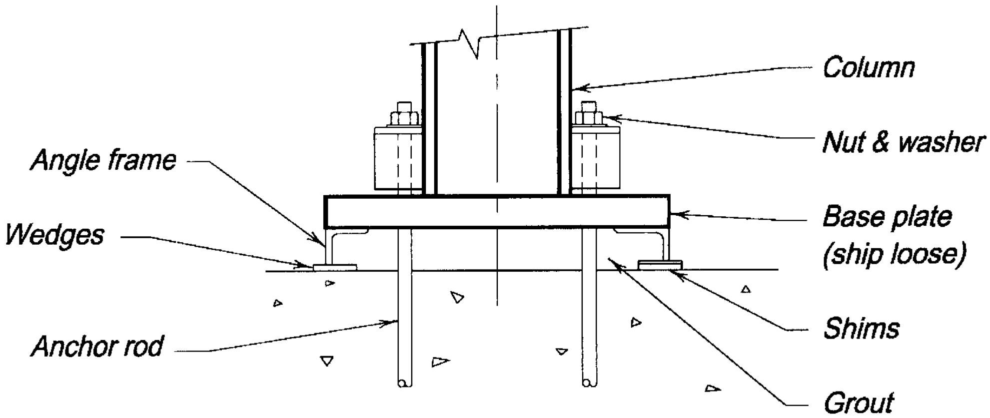


Figure 14–6. Angle-frame leveling.

zontal plates or angles as illustrated in Figure 14–7b. These stiffeners are not usually considered to be part of the column area in bearing on the base plate. The angles preferably should be set back from the column end about $^1/_8$ in. Stiffeners preferably should be set back about 1 in. from the base plate to eliminate a pocket that might prevent drainage and, thus, protect the column and column base plate from corrosion.

For further information, see AISC Design Guide No. 1 *Column Base Plates* (DeWolf and Ricker, 1990).

ANCHOR RODS

Cast-in-place anchor rods, illustrated in Figure 14–8, are generally made from unheaded rod material or headed bolt material. Drilled-in (post-set) anchors can be used for corrective

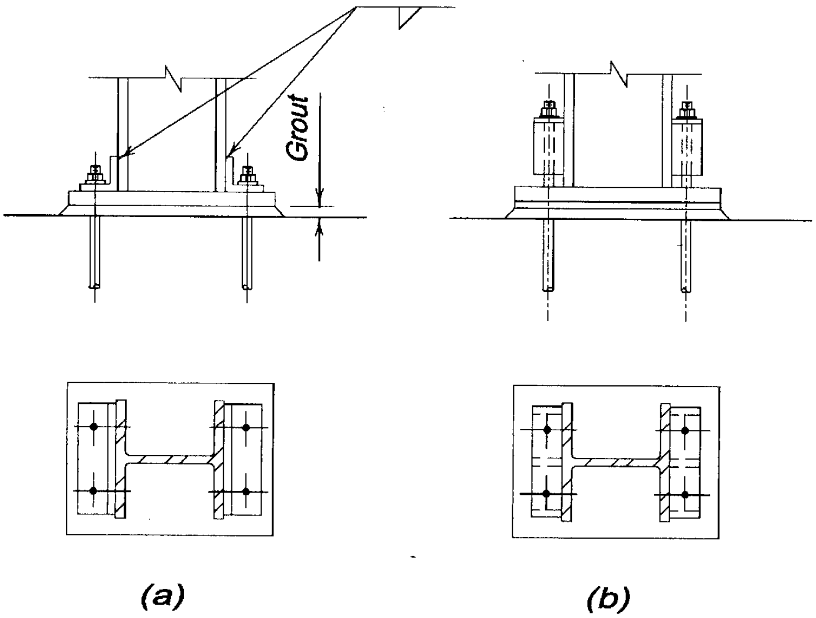


Figure 14–7. Typical column bases for uplift.

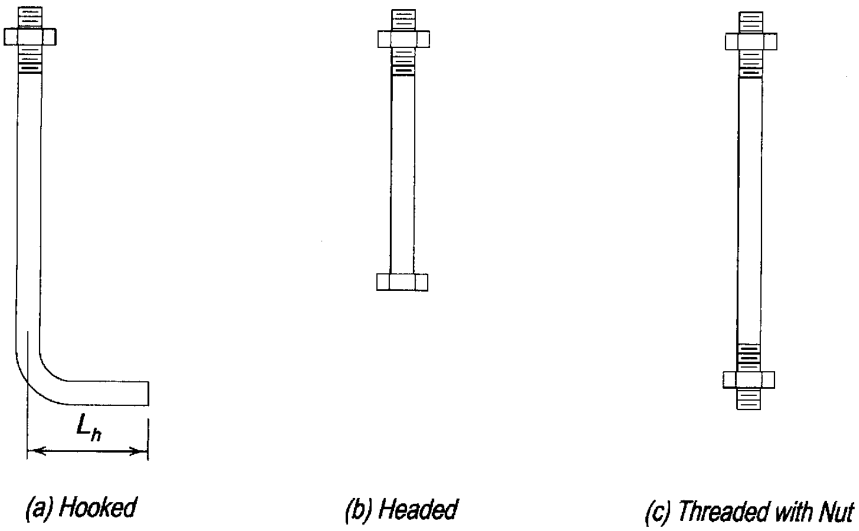


Figure 14–8. Cast-in-place anchor rods.

work or in new work as determined by the owner's designated representative for design and as permitted in the applicable building code. The design of post-set anchors is governed by manufacturer's specifications; see also ACI 349 Appendix B. Post-set anchors that rely upon torque or tension to develop anchorage by wedging action should not be used unless the stability of the column during erection is provided by means other than the post-set anchors.

Minimum Edge Distance and Embedment Length

In general, minimum edge distances, embedment lengths and the design of anchorages into concrete are covered by ACI 318. These provisions include methods to account for edge distance and group action, as does ACI 349. AISC Design Guides 1, 7, and 10 provide additional material on the design of anchor rods in concrete.

In addition to providing the recommended minimum embedment length, anchor rods must extend a distance above the foundation that is sufficient to permit adequate thread engagement of the nut. Adequate thread engagement for anchor rods is identical to the condition described in the RCSC Specification as adequate for steel-to-steel structural joints using high-strength bolts: having the end of the [anchor rod] flush with or outside the face of the nut.

Washer Requirements

Because base plates typically have holes larger than oversized holes to allow for tolerances on the location of the anchor rod, washers are usually furnished from ASTM A36 steel plate. They may be round, square, or rectangular, and generally have holes that are $^1/_{16}$-in. larger than the anchor rod diameter. The thickness must be suitable for the forces to be transferred. Minimum washer sizes are given in Table 14–2.

Hooked Anchor Rods

Hooked anchor rods should be used only for axially loaded members subject to compression only to locate and prevent the displacement or overturning of columns due to erection loads or accidental collisions during erection. Additionally, high-strength steels are not recommended for use in hooked rods since bending with heat may materially affect their strength.

Headed or Threaded and Nutted Anchor Rods

When anchor rods are required for a calculated tensile force, *T*, a more positive anchorage is formed when headed anchor rods, illustrated in Figure 14–8b, are used. With adequate embedment and edge distance, the limit state is either a tensile failure of the anchor rod or the pull-out of a cone of concrete radiating outward from the head (Marsh and Burdette, 1985) as illustrated in Figure 14–9. Marsh and Burdette (1985) showed that the head of the anchor rod usually provides sufficient anchorage and the use of an additional washer or plate does not add significantly to the anchorage. The nut and threading shown in Figure 14–8c is acceptable in lieu of a bolt head. The nut should be welded to the rod to prevent the rod from turning out when the top nut is tightened.

Anchor Rod Nut Installation

The majority of anchorage applications in buildings do not require special anchor rod nut installation procedures or pretension in the anchor rod. The anchor rod nuts should be "drawn down tight" as columns and bases are erected, per ANSI A10.13 Section 9.6. This condition can be achieved by following the same practices as recommended for snug-tightened installation in steel-to-steel bolted joints in the RCSC Specification. That is, most anchor rod nuts can be installed using the full effort of an ironworker with an ordinary spud wrench.

When, in the judgment of the owner's designated representative for design, the performance of the structure will be compromised by excessive elongation of the anchor rods under tensile loads, pretension may be required. Some examples of applications that may require pretension include structures that cantilever from concrete foundations, moment-resisting column bases with significant tensile forces in the anchor rods, or where load reversal might result in the progressive loosening of the nuts on the anchor rods.

When pretensioning of anchor rods is specified, care must be taken in the design of the column base and the embedment of the anchor rod. The shaft of the anchor rod must be free of bond to the encasing concrete so that the rod is free to elongate as it is pretensioned. Also, loss of pretension due to creep in the concrete must be taken into account. Although the design of pretensioned anchorage devices is beyond the scope of this Manual, it should be noted that pretension should not be specified for anchorage devices that have not been properly designed and configured to be pretensioned.

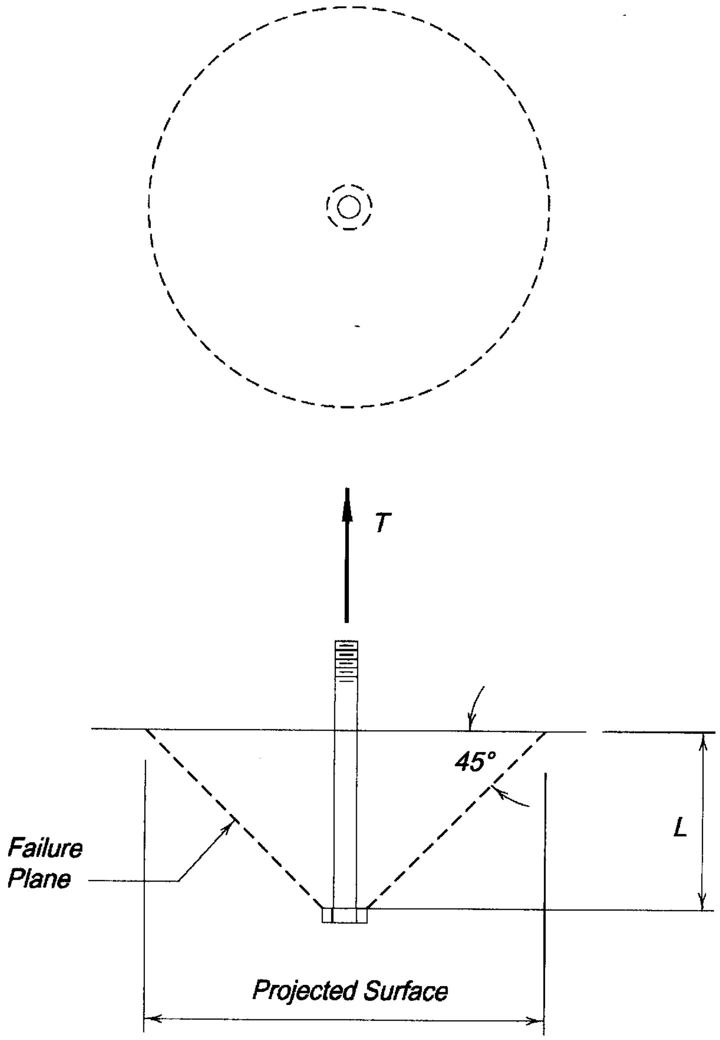


Figure 14–9. Concrete cone subject to pull-out.

COLUMN SPLICES

When the height of a building exceeds the available length of column sections, or when it is economically advantageous to change the column size at a given floor level, it becomes necessary to splice two columns together. Column splices at the final exterior and interior perimeter and at interior openings must be located a minimum of 48 in. above the finished floor to accommodate the attachment of safety cables, except when constructability does not allow. For simplicity and uniformity, other column splices should be located at the same height. Note that column splices placed significantly higher than this are impractical in terms of field assembly.

Fit-Up of Column Splices

From AISC Specification Section M2.6, the ends of columns in a column splice which depend upon contact bearing for the transfer of axial forces must be finished to a common plane by milling, sawing, or other suitable means. In theory, if this were done and the pieces were erected truly plumb, there would be full-contact bearing across the entire surface; this is true in most cases. However, AISC Specification Section M4.4 recognizes that a perfect fit on the entire available surface will not exist in all cases.

A $^1/_{16}$-in. gap is permissible with no requirements for repair or shimming. During erection, at the time of tightening the bolts or depositing the welds, columns will usually be subjected to loads which are significantly less than the design loads. Full-scale tests (Popov and Steven, 1977) which progressed to column failure have demonstrated that subsequent loading to the design loads does not result in distress in the bolts or welds of the splice.

If the gap exceeds $^1/_{16}$ in. but is less than $^1/_4$ in., non-tapered steel shims are required if sufficient contact area does not exist. Mild steel shims are acceptable regardless of the steel grade of the column or bearing material. If required, these shims must be contained, usually with a tack weld, so that they cannot be worked out of the joint.

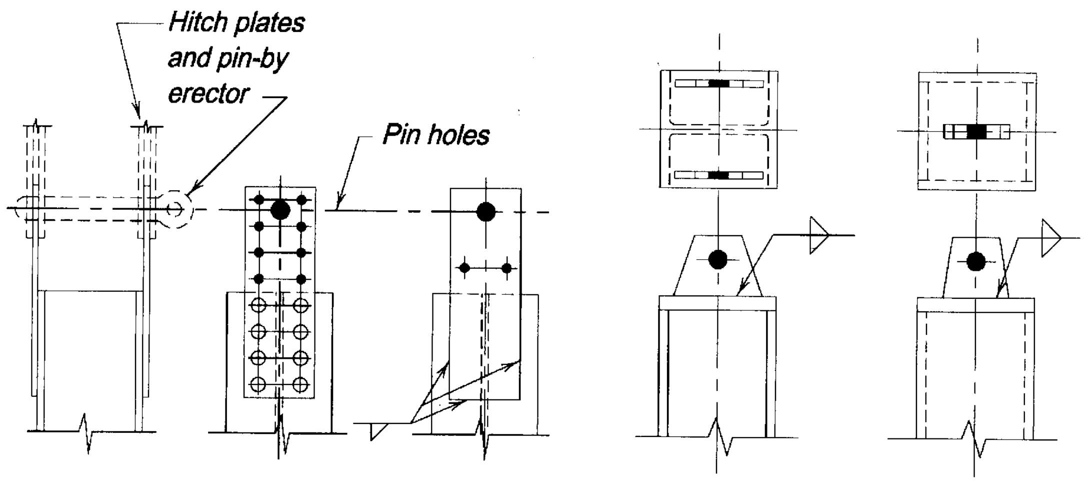
There is no provision in the AISC Specification for gaps larger than $^1/_4$ in. When such a gap exists, an engineering evaluation should be made of this condition based upon the type of loading transferred by the column splice. Tightly driven tapered shims may be required or the required strength may be developed through flange and web splice plates. Alternatively, the gap may be ground or gouged to a suitable profile and filled with weld metal.

Lifting Devices

As illustrated in Figure 14–10, lifting devices are typically used to facilitate the handling and erection of columns. When flange-plated or web-plated column splices are used for W-shape columns, it is convenient to place lifting holes in these flange plates as illustrated in Figure 14–10a. When butt-plated column splices are used, additional temporary plates with lifting holes may be required as illustrated in Figure 14–10b. W-shape column splices which do not utilize web-plated or butt-plated column splices (i.e., groove-welded column splices) may be provided with a lifting hole in the column web as illustrated in Figure 14–10c. While a hole in the column web reduces the cross-sectional area of the column, this reduction will seldom be critical since the column is sized for the loads at the floor below and the splice is located above the floor. Alternatively, auxiliary plates with lifting holes may be connected to the column so that they do not interfere with the welding. Typical column splices for tubes and box-columns are illustrated in Figure 14–10d. Holes in lifting devices may be drilled, reamed, or flame-cut with a mechanically guided torch. In the latter case, the bearing surface of the hole in the direction of the lift must be smooth.

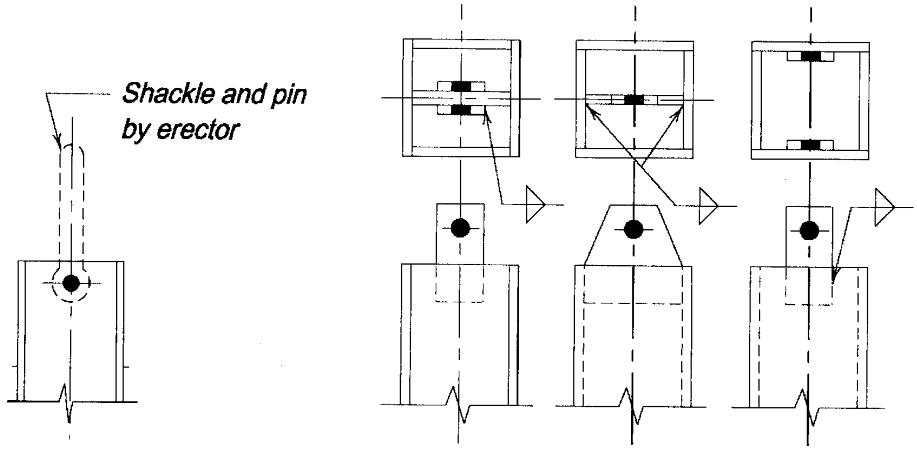
The lifting device and its attachment to the column must be of sufficient strength to support the weight of the column as it is brought from the horizontal position (as delivered) to the vertical position (as erected); the lifting device and its attachment to the column must be adequate for the tensile forces, shear forces, and moments induced during handling and erection.

A suitable shackle and pin are connected to the lifting device while the column is on the ground. The steel erector usually establishes the size and type of shackle and pin to be used in erection and this information must be transmitted to the fabricator prior to detailing. Except for excessively heavy lifting pieces, it is customary to select a single pin and pinhole diameter to accommodate the majority of structural steel members, whether they



(a) W-shape columns, flange-plated column splices with lifting holes

(b) W-shape and box-shape columns. butt-plated column splices with auxiliary lifting plates



(c) W-shape columns, no splice plates, lifting hole in column web

(d) Tubular and box-shape columns, auxiliary lifting plates

Figure 14–10. Lifting devices for columns.

are columns or other heavy structural steel members. The pin is attached to the lifting hook and a lanyard trails to the ground or floor level. After the column is erected and connected, the pin is removed from the device by means of the lanyard, eliminating the need for an ironworker to climb the column. The shackle pin, as assembled with the column, must be free and clear, so that it may be withdrawn laterally after the column has been landed and stabilized.

The safety of the structure, equipment, and personnel is of utmost importance during the erection period. It is recommended that all welds that are used on the lifting devices and stability devices be inspected very carefully, both in the shop and later in the field, for any damage that may have occurred in handling and shipping. Groove welds frequently are inspected with ultrasonic methods (UT) and fillet welds are inspected with magnetic particle (MT) or liquid dye penetrant (PT) methods.

Column Alignment and Stability During Erection

Column splices should provide for safety and stability during erection when the columns might be subjected to wind, construction, and/or accidental loading prior to the placing of the floor system. The nominal flange-plated, web-plated, and butt-plated column splices developed here consider this type of loading.

In other splices, column alignment and stability during erection are achieved by the addition of temporary lugs for field bolting as illustrated in Figure 14–11. The material thickness, weld size, and bolt diameter required are a function of the loading. A conservative resisting moment arm is normally taken as the distance from the compressive toe or flange face to the gage line of the temporary lug. The overturning moment should be checked about both axes of the column. The recommended minimum plate or angle thickness is $^1/_2$ in.; the recommended minimum weld size is $^5/_{16}$ in.; additionally, high-strength bolts are normally used as stability devices.

Temporary lugs are not normally used as lifting devices. Unless required to be removed in the contract documents, these temporary lugs may remain.

Column alignment is provided with centerpunch marks that are useful in centering the columns in two directions.

Force Transfer in Column Splices

As illustrated in Figure 14–12, for the W-shapes most frequently used as columns, the distance between the inner faces of the flanges is constant throughout any given nominal depth group; as the nominal weight per foot increases for each nominal depth, the flange and web thicknesses increase. From AISC Specification Section J7, the available bearing strength, ϕR_n *or* R_n/Ω, of the contact area of a finished surface is determined with

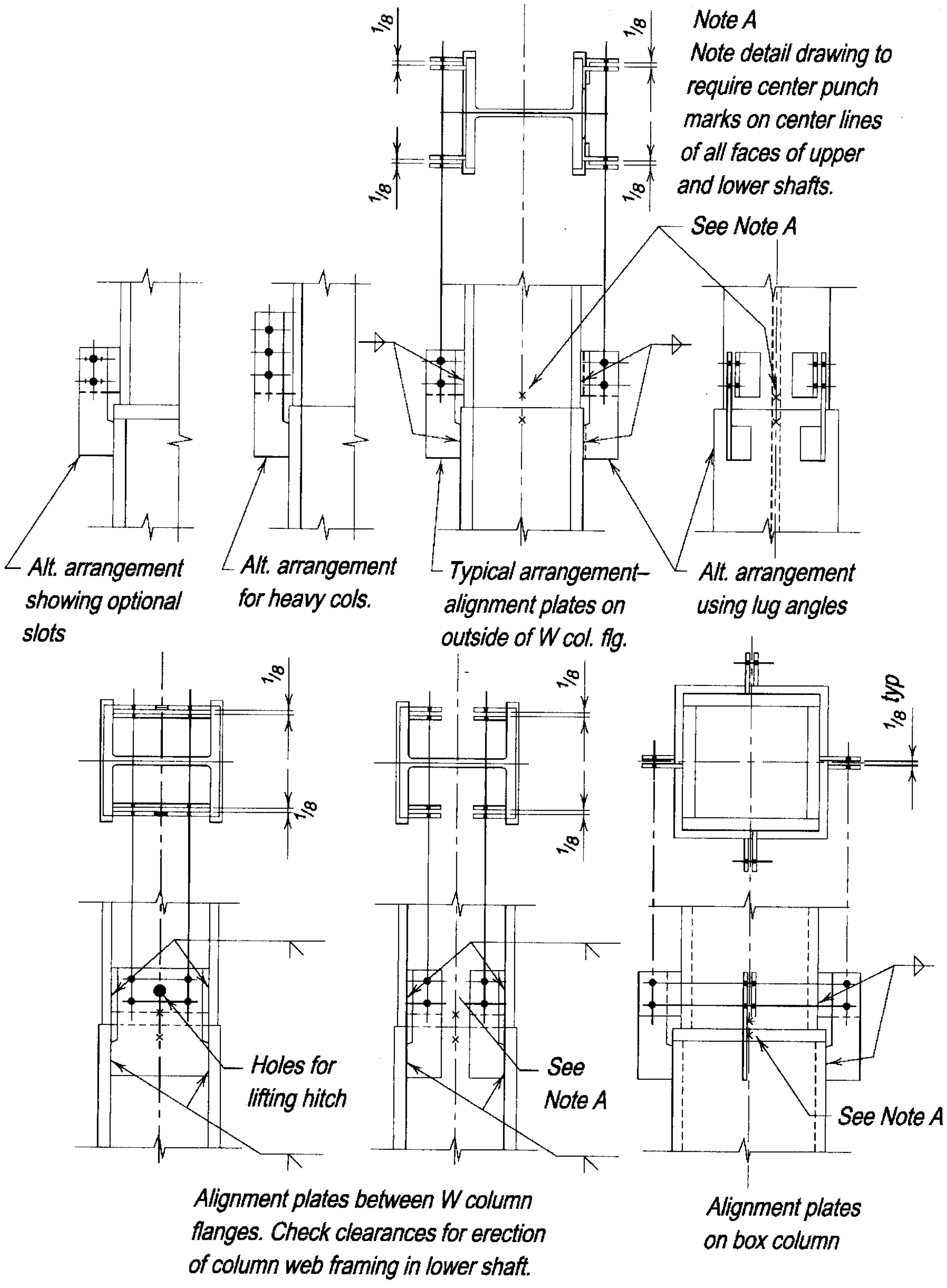
$$R_n = (1.8F_y A_{pb})$$

$$\phi = 0.75 \qquad \Omega = 2.0$$

This bearing strength is much greater than the axial strength of the column and will seldom prove critical in the member design. For column splices transferring only axial forces, complete axial force transfer may be achieved through bearing on finished surfaces; bolts or welds are required by AISC Specification Section J1.4 to be sufficient to hold all parts securely in place.

In addition to axial forces, from AISC Specification Section J1.4, column splices must be proportioned to achieve the required strength in tension, due to the combination of dead load and lateral loads. Note that it is not permissible to use forces due to live load to offset the tensile forces from wind or seismic loads.

For dead and wind loads, if the required strength due to the effect of the dead load is greater than the required strength due to the wind load, the splice is not subjected to tension



Note A
Note detail drawing to require center punch marks on center lines of all faces of upper and lower shafts.

See Note A

Alt. arrangement showing optional slots

Alt. arrangement for heavy cols.

Typical arrangement– alignment plates on outside of W col. flg.

Alt. arrangement using lug angles

Holes for lifting hitch

See Note A

See Note A

Alignment plates between W column flanges. Check clearances for erection of column web framing in lower shaft.

Alignment plates on box column

Figure 14–11. Column stability and alignment devices.

and a nominal splice may be selected from those in Table 14–3. When the required strength due to dead load is less than the required strength due to the wind load, the splice will be subjected to tension and the nominal splices from Table 14–3 are acceptable if the available tensile strength of the splice is greater than or equal to the required strength. Otherwise, a splice must be designed with sufficient area and attachment.

When shear from lateral loads is divided among several columns, the force on any single column is relatively small and can usually be resisted by friction on the contact bearing surfaces and/or by the flange plates, web plates, or butt plates. If the required shear strength exceeds the available shear strength of the column splice selected from Table 14–3, a column splice must be designed with sufficient area and attachment.

The column splices shown in Table 14–3 meet the OSHA requirement for 300 lbs located 18 in. from the column face.

Flange-Plated Column Splices

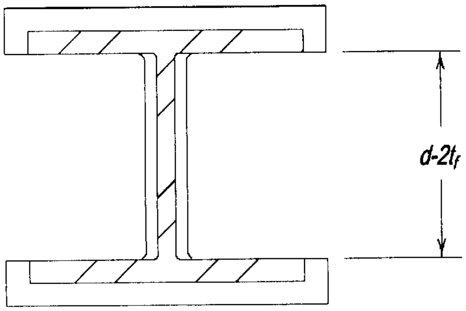
Table 14–3 give typical flange-plated column splice details for W-shape columns. These details are not splice standards, but rather, typical column splices in accordance with AISC Specification provisions and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

Full-contact bearing is always achieved when lighter sections are centered over heavier sections of the same nominal depth group. If the upper column is not centered on the lower column, or if columns of different nominal depths must bear on each other, some areas of the upper column will not be in contact with the lower column. These areas are hatched in Figure 14–13.

When additional bearing area is not required, unfinished fillers may be used. These fillers are intended for "pack-out" of thickness and are usually set back ¼ in. or more from the finished column end. Since no force is transferred by these fillers, only nominal attachment to the column is required.

When additional bearing area is required, fillers finished to bear on the larger column may be provided. Such fillers are proportioned to carry bearing loads at the bearing strength calculated from AISC Specification Section J7 and must be connected to the column to transfer this calculated force.

Although flange plates are shown shop-assembled to the lower column, it is equally acceptable to invert this arrangement and place them on the upper column. This will usually require fills of increased thickness to maintain erection clearances.



Column Size	d-2t_f (in.)
W8×24–67	7.13
W10×33–112	8.86
W12×40–336	10.9
W14×43–808	12.6

Figure 14–12. Distance between flanges for typical W-shape columns.

In Table 14–3, Cases I and II are for all-bolted flange-plated column splices for W-shape columns. Bolts in column splices are usually the same size and type as for other bolts on the column. Bolt spacing, end distance, and edge distances resulting from the plate sizes shown permit the use of $^3/_4$-in. and $^7/_8$-in. bolts in the splice details shown. Larger diameter bolts may require an increase in edge or end distances. Refer to AISC Specification Chapter J. The use of high-strength bolts in bearing-type connections is assumed in all field and shop splices. However, when slotted or oversized holes are utilized, or in splices employing undeveloped fillers over $^1/_4$ in. thick, slip-critical connections may be required; refer to AISC Specification Section J6. For ease of erection, field clearances for lap splices fastened by bolts range from $^1/_8$ in. to $^3/_{16}$ in. under each plate.

Cases IV and V are for all-welded flange-plated column splices for W-shape columns. Splice welds are assumed to be made with E70XX electrodes and are proportioned as required by the AISC Specification provisions. The SAW, GMAW, and FCAW equivalents to E70XX electrodes may be substituted if desired. Field clearance for welded splices are limited to $^1/_{16}$ in. to control the expense of building up welds to close openings. Note that the fillet weld lengths, Y, as compared to the lengths $L/2$, provide 2-in. unwelded distance below and above the column shaft finish line. This provides a degree of flexibility in the splice plates to assist the erector.

Cases VI and VII apply to combination bolted and welded column splices. Since the available strength of the welds will, in most cases, exceed the strength of the bolts, the weld and splice lengths shown may be reduced, if desired, to balance the strength of the fasteners to the upper or lower column, provided that the available strength of the splice is still greater than the required strength of the splice, including erection loading.

Directly Welded Flange Column Splices

Table 14–3 also includes typical directly welded flange column splice details for W-shape and HSS or box-shaped columns. These details are not splice standards, but rather, typical column splices in accordance with AISC Specification provisions and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

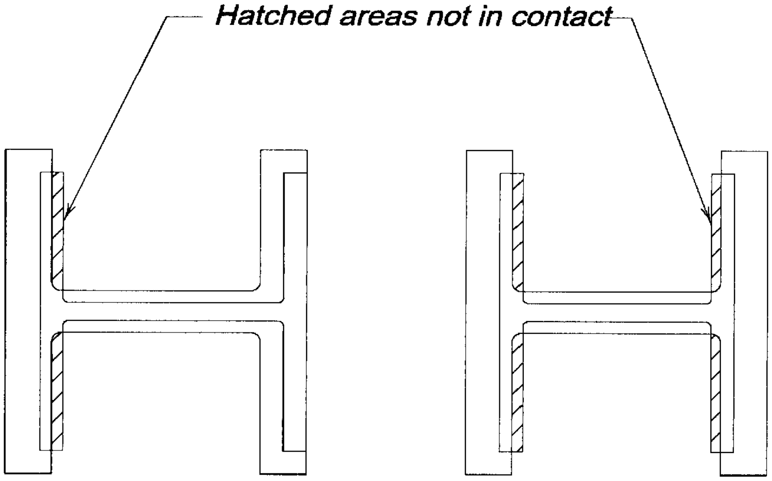


Figure 14–13. Columns not centered or of different nominal depth.

Case VIII applies to W-shape columns spliced with either partial-joint-penetration or complete-joint-penetration groove welds. Case X applies to HSS or box-shaped columns spliced with partial-joint-penetration or complete-joint-penetration groove welds.

Butt-Plated Column Splices

Table 14–3 further includes typical butt-plated column splice details for W-shape and HSS or box-shaped columns. These details are not splice standards, but rather, present typical column splices in accordance with AISC Specification provisions and typical erection requirements. Other splice designs may also be developed. It is assumed in all cases that the lower shaft will be the heavier, although not necessarily the deeper, section.

Butt plates are used frequently on welded splices where the upper and lower columns are of different nominal depths, but may not be economical for bolted splices since fillers cannot be eliminated. Typical butt plates are $1^1/_2$ in. thick for a W8 over W10 splice, and 2-in. thick for other W-shape combinations such as W10 over W12 and W12 over W14. Butt plates which are subjected to substantial bending stresses, such as required on boxed columns, will require a more careful review and analysis. One common method is to assume forces are transferred through the butt plate on a 45° angle and check the thickness obtained for shear and bearing strength. Finishing requirements for butt plates are specified in AISC Specification Section M2.8.

Case III is a combination flange-plated and butt-plated column splice for W-shape columns. Case IX applies to welded butt-plated column splices for W-shape columns. Case XI applies to welded butt-plated column splices for HSS or box-shaped columns. Case XII applies to welded butt-plated column splices between W-shape and HSS or box-shaped columns.

DESIGN CONSIDERATIONS FOR HSS CAP PLATES

The simplest form of attachment to an HSS is to connect the framing member to the top of an HSS. The cap plate serves as a bearing device to transfer the reactions from the framing member into the HSS. The cap plate may also be used to transfer moment into the HSS column. The moment transfer is through a force couple that consists of both compressive and tensile reactions delivered to the cap plate.

Flexural Strength of the Cap Plate

The available strength of the cap plate, in terms of reaction resistance, is determined as ϕR_n or R_n/Ω with

$$R_n = \frac{Bt_c^{\,2}}{4\left(\frac{N_r}{2} + a - \frac{H}{2}\right)} F_{yc}$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where

B = the HSS width, in.
t_c = the cap plate thickness, in.
N_r = the required bearing length for the attached member, in.
a = the distance from the HSS centroid to the end of the attached member, in.

H = the HSS depth, in.
F_{yc} = the specified yield strength of the cap plate, ksi

This equation applies only if the cap plate is subjected to cantilever bending, as shown in Figure 14–14. This occurs when the beam or joist reaction point is outside of the HSS face. If a stiffener is used in the beam and is positioned over the HSS wall, then the equation does not apply, since the cap plate is not subjected to bending. Also if the denominator of the equation results in a negative number, bending of the cap plate can be disregarded.

Compression Yielding and Crippling of the HSS Wall

The available strength of the HSS wall due to compression yielding and compression crippling is determined in accordance with AISC Specification Section K1.6.

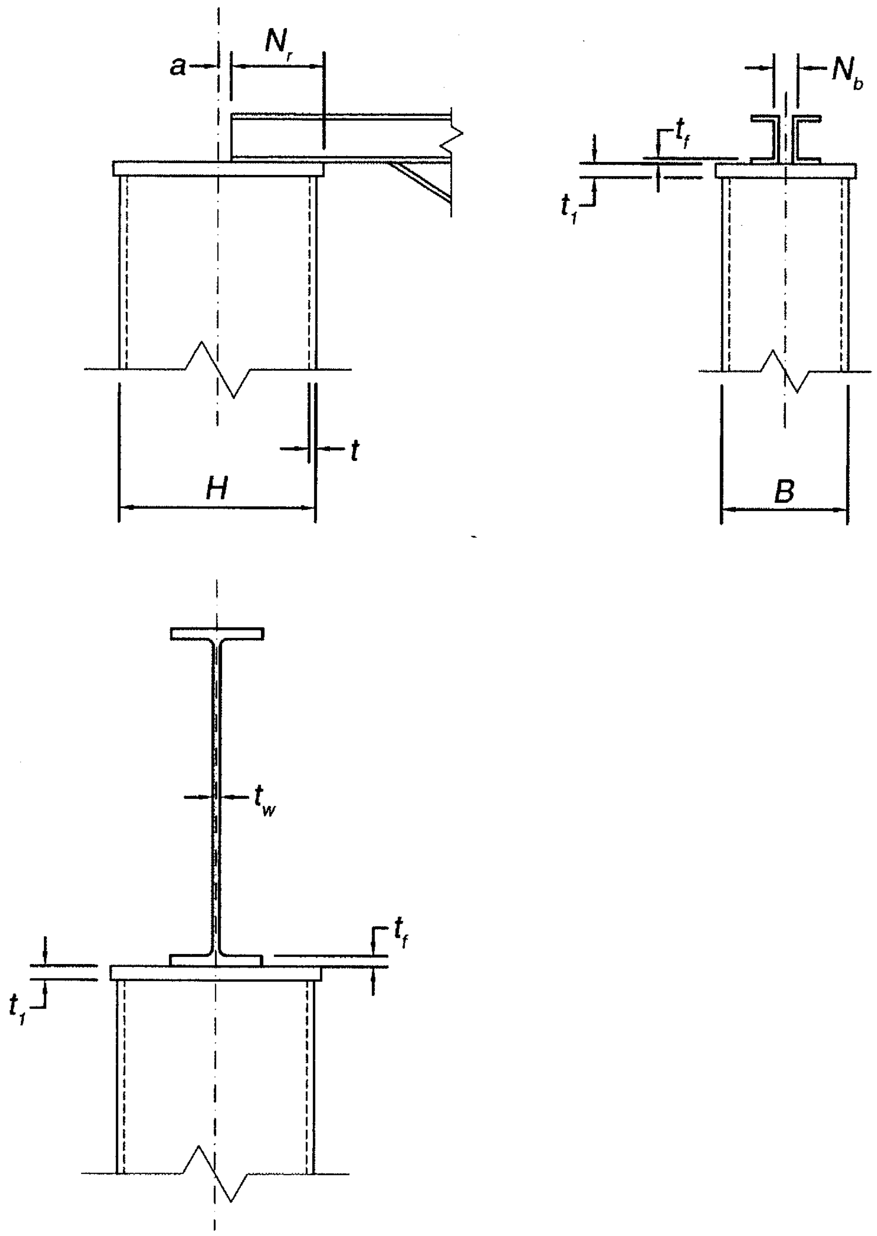


Figure 14–14. Cap plate subject to cantilever bending.

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Table 14–1
Finish Allowances

Size	Thickness (in.)	Add to Fin. One Side (in.)	Add to Fin. Two Sides (in.)
Maximum dimension 24 in. or less	$1\frac{1}{4}$ or less	$\frac{1}{16}$	$\frac{1}{8}$
	over $1\frac{1}{4}$ to 2, incl.	$\frac{1}{8}$	$\frac{1}{4}$
Maximum dimension over 24 in.	$1\frac{1}{4}$ or less	$\frac{1}{8}$	$\frac{1}{4}$
	over $1\frac{1}{4}$ to 2, incl.	$\frac{3}{16}$	$\frac{3}{8}$
56 in. wide or less	over 2 to $7\frac{1}{2}$, incl.	$\frac{1}{4}$	$\frac{3}{8}$
	over $7\frac{1}{2}$ to 10, incl.	$\frac{1}{2}$	$\frac{5}{8}$
	over 10 to 15, incl.	$\frac{3}{4}$	$\frac{7}{8}$
Over 56 in. wide to 72 in. wide	over 2 to 6, incl.	$\frac{1}{4}$	$\frac{3}{8}$
	over 6 to 10, incl.	$\frac{1}{2}$	$\frac{5}{8}$
	over 10 to 15, incl.	$\frac{3}{4}$	$\frac{7}{8}$

Table 14–2
Recommended Maximum Sizes for Anchor-Rod Holes in Base Plates

Anchor Rod Diameter, in.	Max. Hole Diameter, in.	Min. Washer Size, in.	Min. Washer Thickness	Anchor Rod Diameter, in.	Hole Diameter, in.	Min. Washer Size, in.	Min. Washer Thickness
$\frac{3}{4}$	$1\frac{5}{16}$	2	$\frac{1}{4}$	$1\frac{1}{2}$	$2\frac{5}{16}$	$3\frac{1}{2}$	$\frac{1}{2}$
$\frac{7}{8}$	$1\frac{9}{16}$	$2\frac{1}{2}$	$\frac{5}{16}$	$1\frac{3}{4}$	$2\frac{3}{4}$	4	$\frac{5}{8}$
1	$1\frac{13}{16}$	3	$\frac{3}{8}$	2	$3\frac{1}{4}$	5	$\frac{3}{4}$
$1\frac{1}{4}$	$2\frac{1}{16}$	3	$\frac{1}{2}$	$2\frac{1}{2}$	$3\frac{3}{4}$	$5\frac{1}{2}$	$\frac{7}{8}$

Notes:
1. Circular or square washers meeting the washer size are acceptible.
2. Clearance must be considered when choosing an appropriate anchor rod hole location, noting effects such as the position of the rod in the hole with respect to the column, weld size and other interferences.
3. When base plates are less than $1\frac{1}{4}$ in. thick, punching of holes may be an economical option. In this case, $\frac{3}{4}$-in. anchor rods and $1\frac{1}{16}$-in. diameter punched holes may be used with ASTM F844 (USS Standard) washers in place of fabricated plate washers.

Table 14–3

Typical Column Splices

Case I:

All-bolted flange-plated column splices between columns with depth d_u and d_l nominally the same.

Column Size	Gage g_u or g_l	Flange Plates			
		Type	Width	Thk.	Length
W14×455 to 730	13½	1	16	¾	1′ 6½
257 to 426	11½	1	14	⅝	1′ 6½
145 to 233	11½	1	14	½	1′ 6½
90 to 132	11½	2	14	⅜	1′ 0½
43 to 82	5½	2	8	⅜	1′ 0½
W12×120 to 336	5½	2	8	⅝	1′ 0½
40 to 106	5½	2	8	⅜	1′ 0½
W10×33 to 112	5½	2	8	⅜	1′ 0½
W8×31 to 67	5½	2	8	⅜	1′ 0½
24 & 28	3½	2	6	⅜	1′ 0½

Gages shown may be modified if necessary to accommodate fittings elsewhere on the column.

Case I-A: $d_l = (d_u + 1/4$ in.) to $(d_u + 5/8$ in.)	Flange plates: Select g_u for upper column; select g_l and flange plate dimensions for lower columns (see table above). Fillers: None. Shims: Furnish sufficient strip shims 2½×⅛ to provide 0 to $1/16$-in. clearance each side.
Case I-B: $d_l = (d_u - 1/4$ in.) to $(d_u + 1/8$ in.)	Flange plates: Same as Case I-A. Fillers (shop bolted under flange plates): Select thickness as ⅛-in. for $d_l = d_u$ and $d_l = (d_u + 1/8$ in.) or as ¼-in. for $d_l = (d_u - 1/8$ in.) and $d_l = (d_u - 1/4$ in.) Select width to match flange plate and length as 0′ 9 for Type 1 or 0′ 6 for Type 2. Shims: Same as Case I-A.
Case I-C: $d_l = (d_u + 3/4$ in.) and over.	Flange plates: Same as Case I-A. Fillers (shop bolted to upper column): Select thickness as $(d_l - d_u) / 2$ minus ⅛ in. or $3/16$ in., whichever results in ⅛-in. multiples of filler thickness. Select width to match flange plate, but not greater than upper column flange width. Select length as 1′ 0 for Type 1 or 0′ 9 for Type 2. Shims: Same as Case I-A.

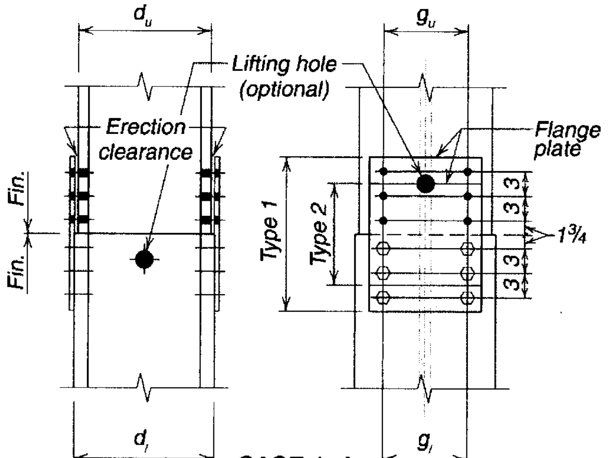
For lifting devices, see Figure 14-11.

Table 14–3 (continued)

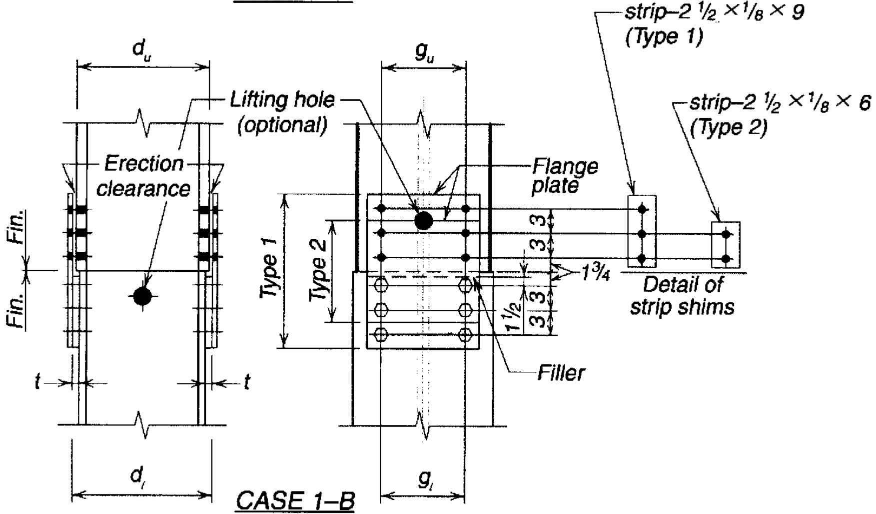
Typical Column Splices

Case I:

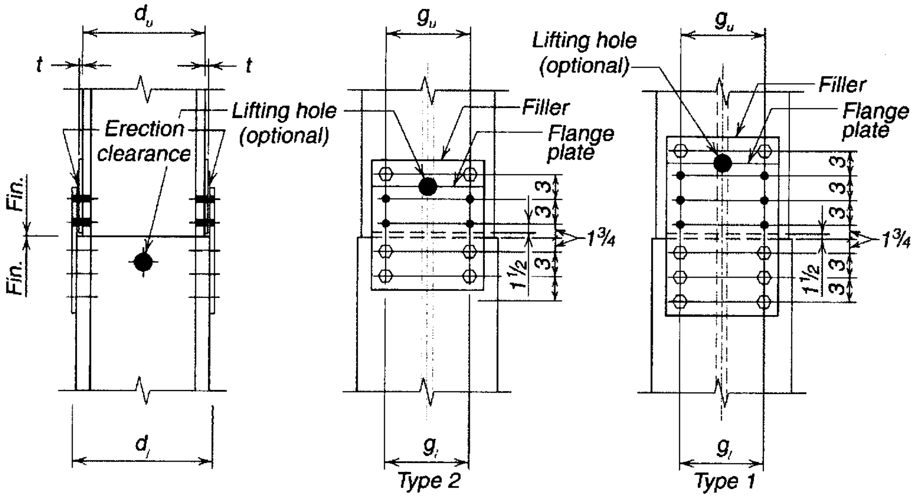
All-bolted flange-plated column splices between columns with depth d_u and d_l nominally the same.



CASE 1–A



CASE 1–B



CASE 1–C

Table 14–3 (continued)

Typical Column Splices

Case II:

All-bolted flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

Fillers on upper column developed for bearing on lower column.

Flange plates: Same as Case I-A.

Fillers (shop bolted to upper column): Select thickness as $(d_l - d_u)/2$ minus ⅛-in. or 3/16-in., whichever results in ⅛-in. multiples of filler thickness. Select bolts through fillers (including bolts through flange plates) on each side to develop bearing strength of the filler. Select width to match flange plate, but not greater than upper column flange width unless required for bearing strength. Select length as required to accommodate required number of bolts.

Shims: Same as Case I-A.

Table 14–3 (continued)

Typical Column Splices

Case III:

All-bolted flange-plated and butt-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

Fillers on upper column developed for bearing on lower column.

Column Size	Gage g_u or g_l	Flange Plates			
		Type	Width	Thk.	Length
W14×455 to 730	13½	1	16	¾	1′ 8½
257 to 426	11½	1	14	⅝	1′ 8½
145 to 233	11½	1	14	½	1′ 8½
90 to 132	11½	2	14	⅜	1′ 2½
43 to 82	5½	2	8	⅜	1′ 2½
W12×120 to 336	5½	2	8	⅝	1′ 2½
40 to 106	5½	2	8	⅜	1′ 2½
W10×33 to 112	5½	2	8	⅜	1′ 2½
W8×31 to 67	5½	2	8	⅜	1′ 2
24 & 28	3½	2	8	⅜	1′ 2

Gages shown may be modified if necessary to accommodate fittings elsewhere on the column.

Flange plates: Select g_u for upper column, select g_l and flange plate dimensions for lower column (see table above).

Fillers (shop bolted to upper column): Same as Case I-C.

Shims: Same as Case I-A.

Butt plate: Select thickness as 1½-in. for W8 upper column or two inches for others. Select width the same as upper column and length as d_l – ¼ in.

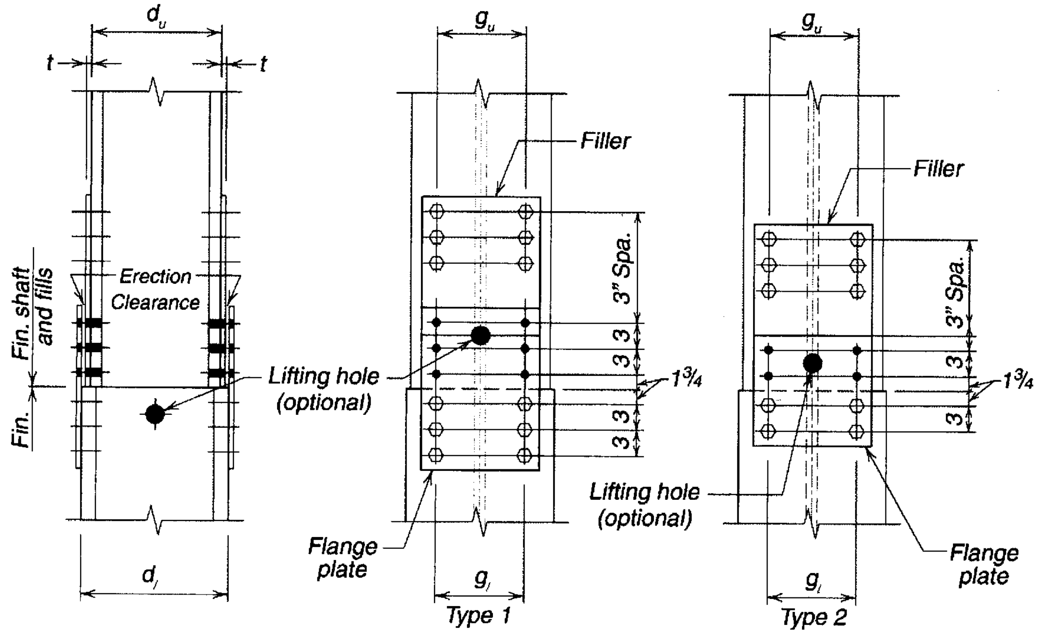
For lifting devices, see Figure 14–11.

Table 14–3 (continued)

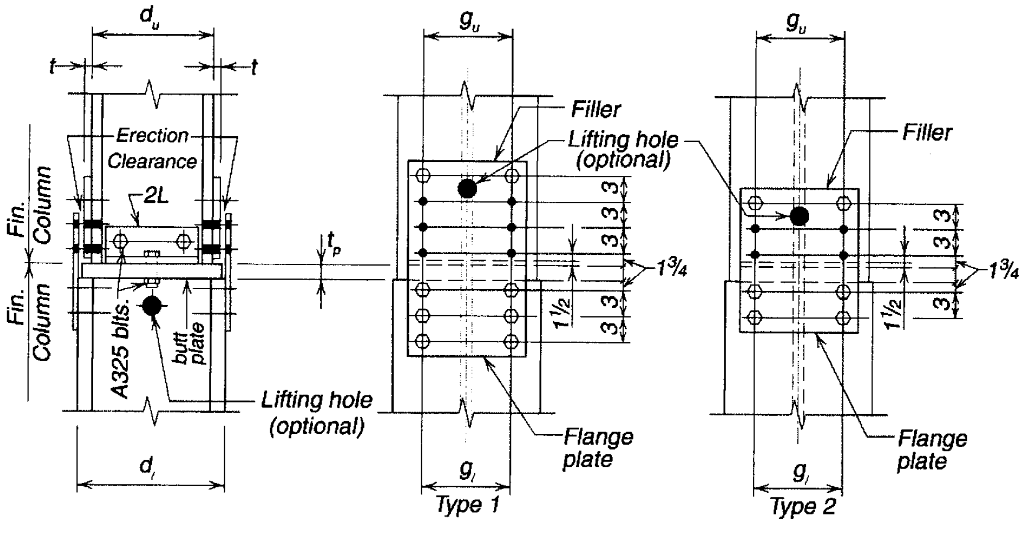
Typical Column Splices

Case II and III:

All-bolted flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.



CASE II



CASE III

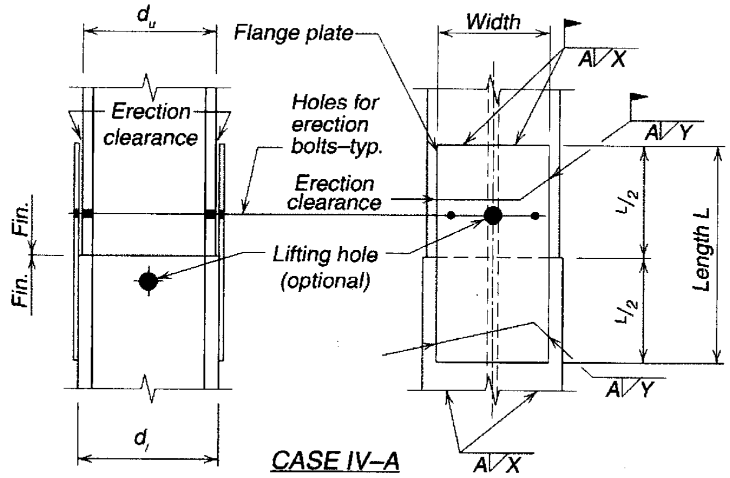
Table 14–3 (continued)
Typical Column Splices
Case IV:
All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same.

Column Size	Flange Plate			Welds			Minimum Space for Welding	
	Width	Thk.	Length L	Size A	Length			
					X	Y	M	N
W14×455 & over	14	5/8	1′-6	1/2	5	7	13/16	11/16
311 to 426	12	5/8	1′-4	1/2	4	6	13/16	11/16
211 to 283	12	1/2	1′-4	3/8	4	6	11/16	9/16
90 to 193	12	3/8	1′-4	5/16	4	6	5/8	1/2
61 to 82	8	3/8	1′-4	5/16	3	6	5/8	1/2
43 to 53	6	5/16	1′-2	1/4	2	5	9/16	7/16
W12×120 to 336	8	1/2	1′-4	3/8	3	6	11/16	9/16
53 to 106	8	3/8	1′-4	5/16	3	6	5/8	1/2
40 to 50	6	5/16	1′-2	1/4	2	5	9/16	7/16
W10×49 to 112	8	3/8	1′-4	5/16	3	6	5/8	1/2
33 to 45	6	5/16	1′-2	1/4	2	5	9/16	7/16
W8×31 to 67	6	3/8	1′-2	5/16	2	5	5/8	1/2
24 & 28	5	5/16	1′-0	1/4	2	4	9/16	7/16

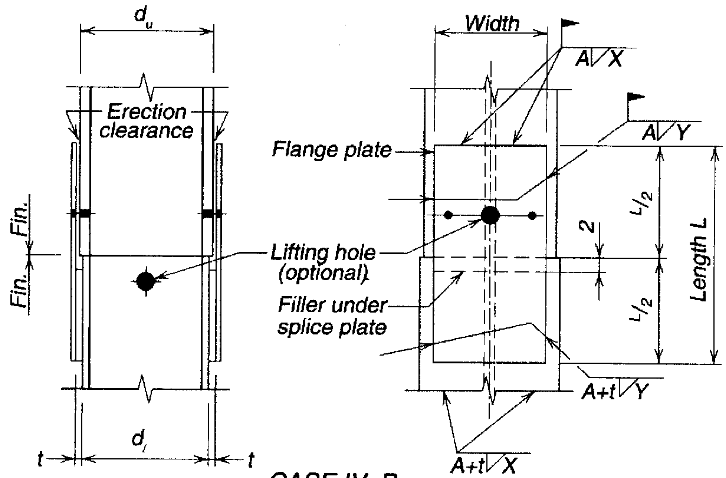
Case IV-A: $d_l = (d_u + 1/8)$	Flange plates: Select flange-plate width and length and weld lengths for upper (lighter) column; select flange-plate thickness and weld size for lower (heavier) column. Fillers: None.
Case IV-B: $d_l = (d_u - 1/4$ in.) to d_u	Flange plates: Same as Case IV-A, except use weld size $A + t$ on lower column. Fillers (undeveloped on lower column, shop welded under flange plates): Select thickness t as $(d_l - d_u)/2 + 1/16$ in. Select width to match flange plate and length as $L/2 - 2$ in.
Case IV-C: $d_l = (d_u + 1/4$ in.) to $(d_u + 1/2$ in.)	Flange plates: Same as Case IV-A, except use weld size $A + t$ on upper column. Fillers (undeveloped on upper column, shipped loose): Select thickness t as $(d_l - d_u)/2 - 1/16$ in. Select width to match flange plate and length as $L/2 - 2$ in.

For lifting devices, see Figure 14–11.

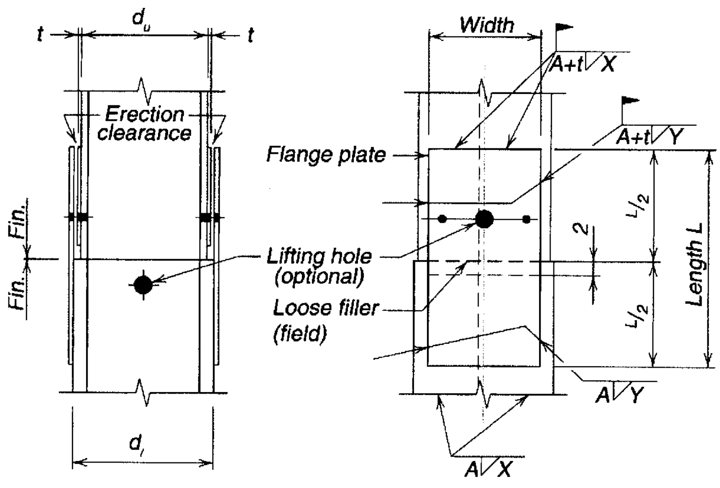
Table 14–3 (continued)
Typical Column Splices
Case IV:
All-bolted flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.



CASE IV–A



CASE IV–B



CASE IV–C

Table 14–3 (continued)

Typical Column Splices

Case IV:

All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same.

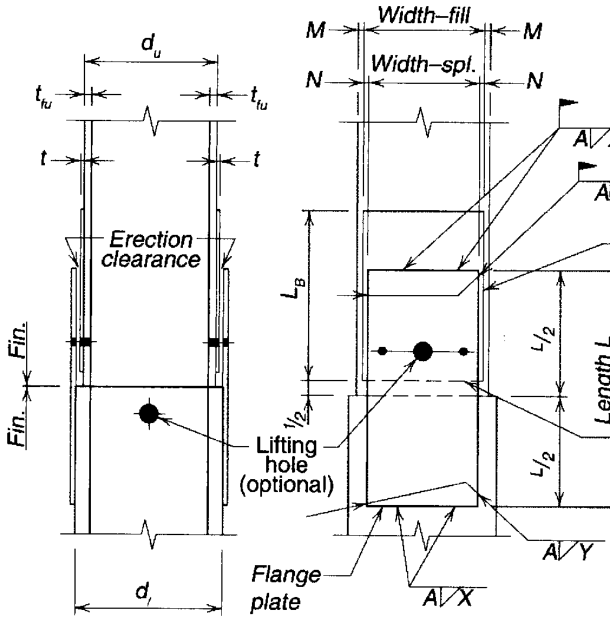
Case IV-D: $d_l = (d_u + {}^5\!/_8$ in.) and over Filler width less than upper column flange width.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (developed on upper column, shop welded to upper column): Select thickness t as $(d_l - d_u) / 2 - {}^1\!/_{16}$ in. Select weld size B from AISC Specification; $\leq {}^5\!/_{16}$-in. preferred. Select weld length L_B such that $L_B \geq A(X + Y) / B \geq (L / 2 + 1$ in.). Select filler width greater than flange plate width + $2N$ but less than upper column flange width − $2M$. Select filler length as L_B, subject to Note 2.
Case IV-E: $d_l = (d_u + {}^5\!/_8$ in.) and over Filler width greater than upper column flange width. Use this case only when M or N in Case IV-D are inadequate for welds B and A.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (developed on upper column, shop welded to upper column): Select thickness t as $(d_l - d_u) / 2 - {}^1\!/_{16}$ in. Select weld size B from AISC Specification; $\leq {}^5\!/_{16}$-in. preferred. Select weld length L_B such that $L_B \geq A(X + Y) / B \geq (L / 2 + 1$ in.). Select filler width as the larger of the flange plate width + $2N$ and the upper column flange width + $2M$, rounded to the next higher ${}^1\!/_4$-in. increment. Select filler length as L_B subject to Note 2.

Table 14–3 (continued)

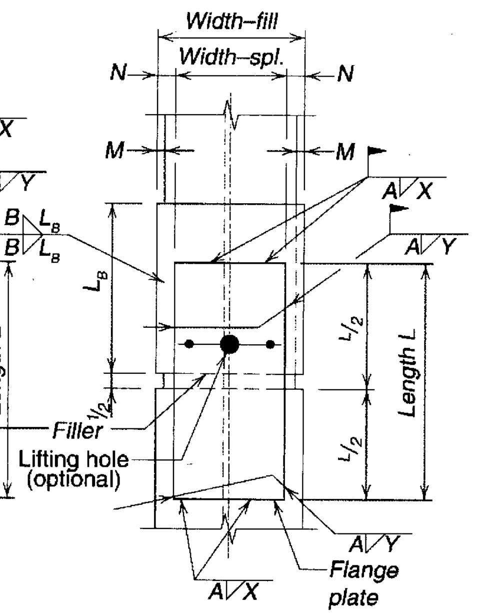
Typical Column Splices

Case IV:

All-welded flange-plated column splices between columns with depths d_u and d_l nominally the same.



CASE IV–D



CASE IV–E

Note 1:

Where welds fasten flange plates to developed fillers, or developed fillers to column flanges (Cases IV-E and V-B), use the table to the right to check minimum fill thickness for balanced fill and weld shear strength. Assume that an E70XX weld with $A = \frac{1}{2}$, $X = 4$, and $Y = 6$ is to be used at full strength on an A36 fill ¼-in. thick. Since this table shows that the minimum fill thickness to develop this ½-in. weld is 0.51 in., the ¼-in. fill will be overstressed. A balanced condition is obtained by multiplying the length $(X + Y)$ by the ratio of the minimum to the actual thickness of fill, thus:

Weld A E70XX	Minimum Fill Thickness for Balanced Weld and Plate Shear	
	F_y	
	36	50
¼	0.26	0.19
5⁄16	0.32	0.23
⅜	0.38	0.28
7⁄16	0.45	0.33
½	0.51	0.37

$$(4 + 6) \times \frac{0.51}{0.25} = 20.4$$

use $(X + Y) = 20\frac{1}{2}$-in.

Placing this additional increment of $(X + Y)$ can be done by making weld lengths X continuous across the end of the splice plate and by increasing Y (and therefore the plate Length) if required.

Note 2:

If fill length, based on L_B, is excessive, place weld of size B across one or both ends of fill and reduce L_B accordingly, but not to less than $(L/2 + 1)$. Omit return welds in Cases IV-E and V-B.

Table 14–3 (continued)

Typical Column Splices

Case V:

All-welded flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

Case V-A: Fillers on upper column developed for bearing on lower column. Filler width less than upper column flange width.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (shop welded to upper column): Select thickness as $(d_l - d_u)/2 - \frac{1}{16}$ in. Select weld size B from AISC Specification; $\leq \frac{5}{16}$ in. preferred. Select weld length L_B to develop bearing strength of the filler but not less than $(L/2 + 1\frac{1}{2}$ in.). Select filler width greater than the flange plate width + $2N$ but less than the upper column flange width − $2M$. See Case IV for M and N.
Case V-B: Same as Case V-A except filler width is greater than upper column flange width. Use this case only when M or N in Case V-A are inadequate for weld A, or when additional filler bearing area is required.	Flange plates: Same as Case IV-A, except see Note 1. Fillers (shop welded to upper column): Select thickness as $(d_l - d_u)/2 - \frac{1}{16}$ in. Select weld size B from AISC Specification; $\leq \frac{5}{16}$ in. preferred. Select weld length L_B to develop bearing strength of the filler but not less than $(L/2 + 1\frac{1}{2}$ in.). Select filler width as the larger of the flange plate width + $2N$ and the upper column flange width + $2M$, rounded to the next higher $\frac{1}{4}$ in. increment. Filler length as L_B, subject to Note 3.

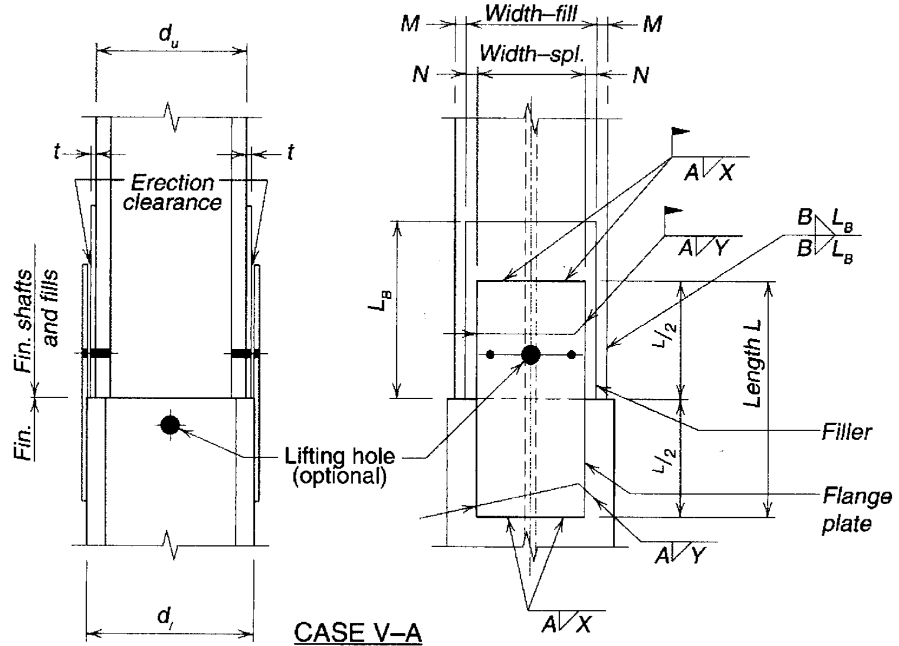
Note 3:
If fill length, based on L_B, is excessive, place weld of size B across end of fill and reduce L_B by one-half of such additional weld length, but not to less than $(L/2 + 1\frac{1}{2})$. Omit return welds in Case V-B.

Table 14–3 (continued)

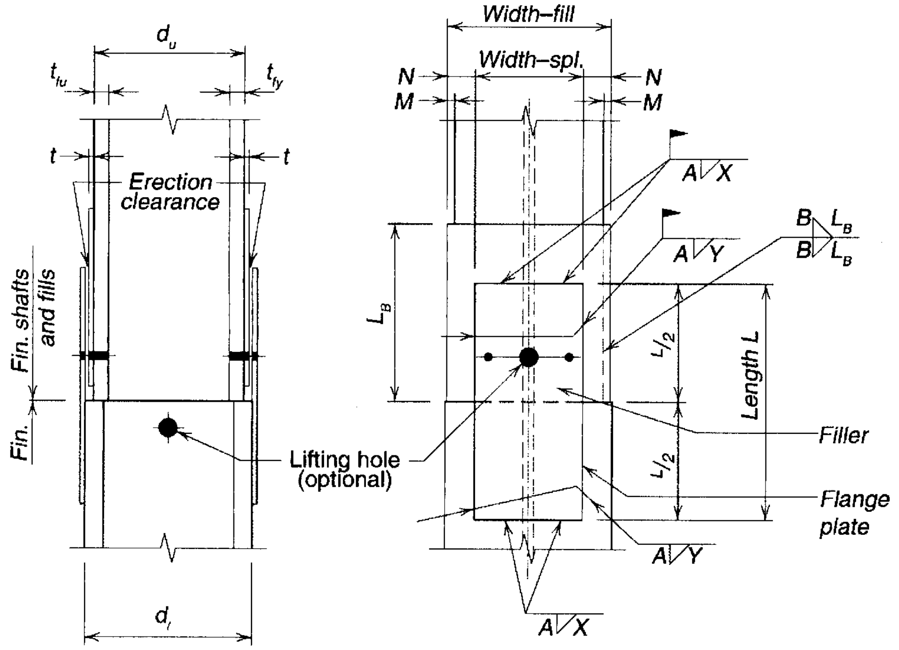
Typical Column Splices

Case V:

All-welded flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l.



CASE V–A



CASE V–B

Table 14–3 (continued)

Typical Column Splices

Case VI:

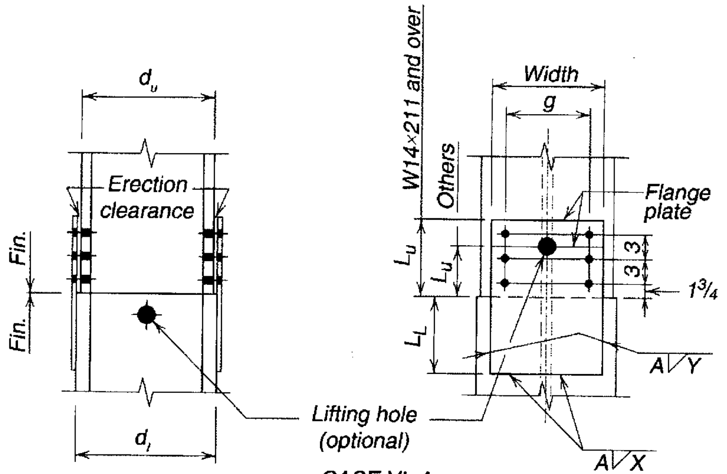
Combination bolted and welded column splices between columns with depths d_u and d_l nominally the same.

Column Size	Flange Plate				Bolts		Welds		
	Width	Thk.	Length		No. of Rows	Gage g	Size A	Length	
			L_U	L_L				X	Y
W14×455 & over	14	⅝	9¼	9	3	11½	½	5	7
311 to 426	12	⅝	9¼	8	3	9½	½	4	6
211 to 283	12	½	9¼	8	3	9½	⅜	4	6
90 to 193	12	⅜	6¼	8	2	9½	5/16	4	6
61 to 82	8	⅜	6¼	8	2	5½	5/16	3	6
43 to 53	6	5/16	6¼	7	2	3½	¼	2	5
W12×120 to 336	8	½	6¼	8	2	5½	⅜	3	6
53 to 106	8	⅜	6¼	8	2	5½	5/16	3	6
40 to 50	6	5/16	6¼	7	2	3½	¼	2	5
W10×49 to 112	8	⅜	6¼	8	2	5½	5/16	3	6
33 to 45	6	5/16	6¼	7	2	3½	¼	2	5
W8×31 to 67	6	⅜	6¼	7	2	3½	5/16	2	5
24 & 28	5	5/16	6¼	6	2	3½	¼	2	4

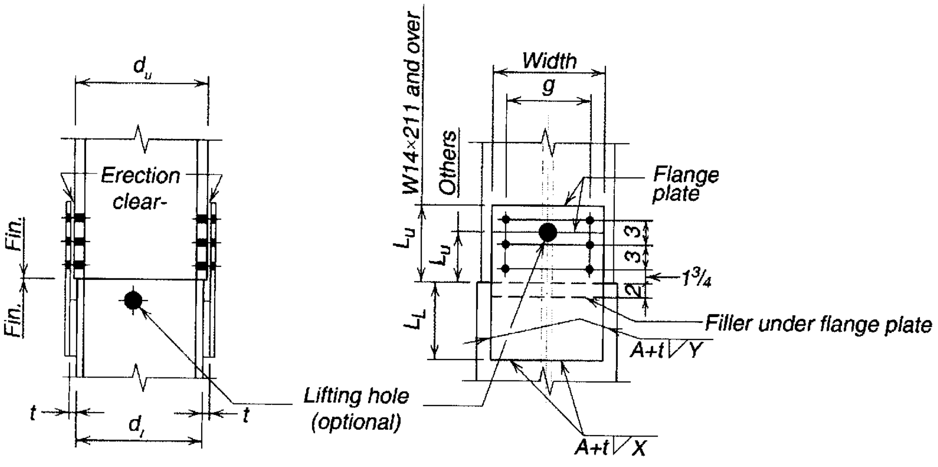
Gages shown may be modified if necessary to accommodate fittings elsewhere on the columns.

Case VI-A: $d_l = (d_u + 1/4 \text{ in.})$ to $(d_u + 5/8 \text{ in.})$	Flange plates: Select flange plate width, bolts, gage and length L_U for upper column; select flange plate thickness, weld size A, weld lengths X and Y, and length L_L for lower column. Total flange plate length is $L_U + L_L$ (see table above). Fillers: None. Shims: Furnish sufficient strip shims 2½×⅛ to obtain 0 to 1/16-in. clearance on each side.
Case VI-B: $d_l = (d_u - 1/4 \text{ in.})$ to $(d_u + 1/8 \text{ in.})$	Flange plates: Same as Case VI-A, except use weld size $A + t$ on lower column. Fillers (shop welded to lower column under flange plate): Select thickness t as ⅛-in. for for $d_l = d_u$ and $d_l = (d_u + 1/8 \text{ in.})$ or as 3/16-in. for $d_l = (d_u - 1/8 \text{ in.})$ and $d_l = (d_u - 1/4 \text{ in.})$. Select width to match flange plate and length as $L_L - 2$ in. Shims: Same as Case VI-A.
Case VI-C: $d_l = (d_u + 3/4 \text{ in.})$ and over	Flange plates: Same as Case VI-A. Fillers (shop welded to upper column): Select thickness t as $(d_l - d_u) / 2$ minus ⅛-in. or 3/16-in., whichever results in ⅛-in. multiples of fill thickness. Select weld size B as minimum size from AISC Specification Section J2. Select weld length as $L_U - 1/4$ in. Select filler width as flange plate width and filler length as $L_U - 1/4$-in. Shims: Same as Case VI-A.

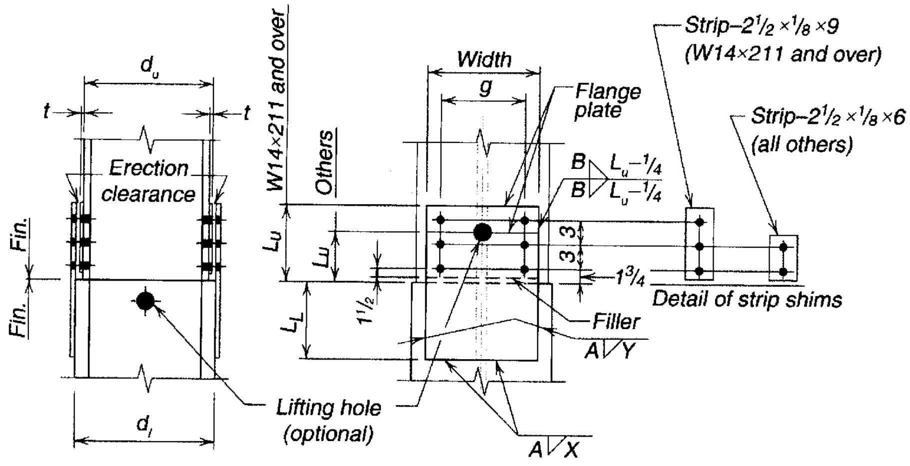
Table 14–3 (continued)
Typical Column Splices
Case VI:
Combination bolted and welded column splices between columns with depths d_u and d_l nominally the same.



CASE VI-A



CASE VI-B



CASE VI-C

Table 14–3 (continued)
Typical Column Splices
Case VII:
Combination bolted and welded flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l. Fillers developed for bearing.

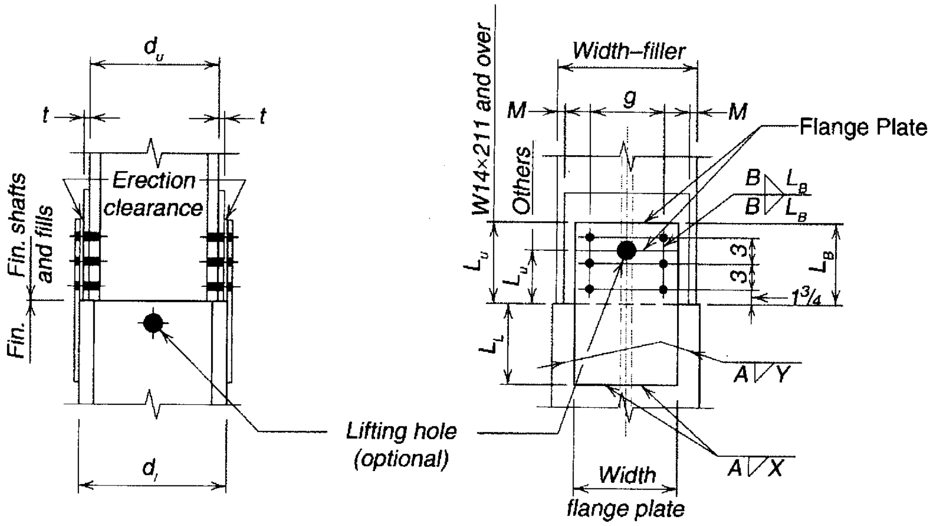
Case VII-A: Fillers of width less than upper column flange width.	Flange plates: Same as Case VI-A. Fillers (shop welded to upper column): Select filler thickness t as $(d_l - d_u)/2$ minus ⅛-in. or $^3\!/_{16}$-in., whichever results in ⅛-in. multiples of filler thickness. Select weld size B from AISC Specification; $\leq ^5\!/_{16}$-in. preferred. Select weld length L_B to develop bearing strength of filler. Select filler width not less than flange plate width but not greater than upper column flange width $-2M$ (see Case IV). Select filler length as L_B, subject to Note 4.
Case VII-B: Filler of width greater than upper column flange width. Use Case VII-B only when fillers must be widened to provide additional bearing area.	Flange plates: Same as Case VI-A. Fillers (shop welded to upper columns): Same as Case VII-A except select filler width as upper column flange width $+ 2M$ (see Case IV) rounded to the next larger ½-in. increment.
Note 4: If fill length based on L_B is excessive, place weld of size B across end of fill and reduce L_B by one-half of such additional weld length, but not less than L_U. Omit return welds, Case VII-B.	

Table 14–3 (continued)

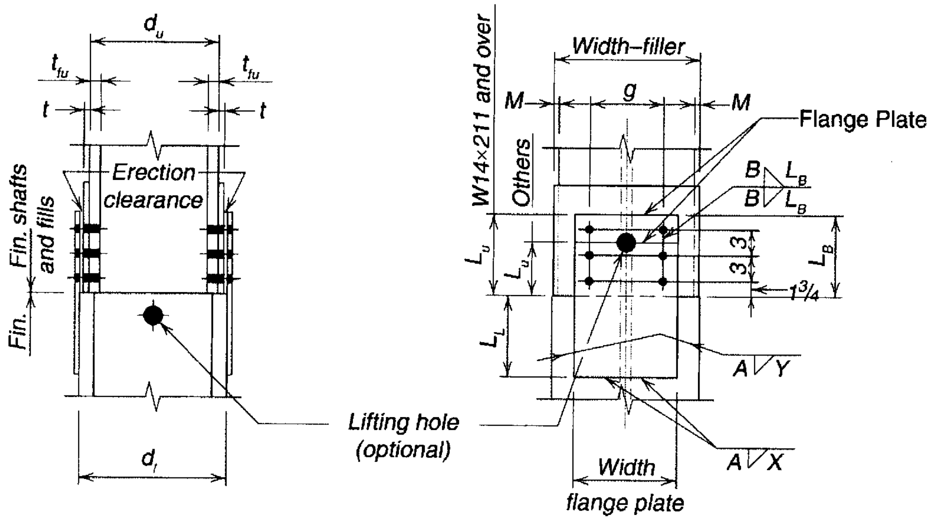
Typical Column Splices

Case VII:

Combination bolted and welded flange-plated column splices between columns with depth d_u nominally two inches less than depth d_l. Fillers developed for bearing.



CASE VII–A



CASE VII–B

Table 14–3 (continued)

Typical Column Splices

Case VIII:

Directly welded flange column splices between columns with depths d_u and d_l nominally the same.

These types of splices exhibit versatility. The flanges may be partial-joint-penetration welded as in Cases VIIIA and VIIIB, or complete-joint-penetration welded as in Cases VIIIC, VIIID, and VIIIE The webs may be spliced using the channel(s) as shown in Cases VIIIA, VIIIB, VIIIC, and VIIID, or complete-joint-penetration welded as shown in Case VIIIE. The use of a channel or channels at the web splice provides a higher degree of restraint during the erection phase than does a plate or plates. The use of partial-joint-penetration flange welds provide greater stability during the erection phase than do complete-joint-penetration welds.

The adequacy of any splice arrangement must be confirmed by the user. This is especially true in regions where high winds are prevalent or when the concentrated weight of the fabricated column is significantly off its centerline. When using partial-joint-penetration flange welds, a land width of ¼-in. or greater should be used. The weld sizes are based on the thickness of the thinner column flange, regardless of whether it is the upper or lower column.

When column flange thicknesses are less than ½-in. it may be more efficient to use flange splice plates as shown in previous cases.

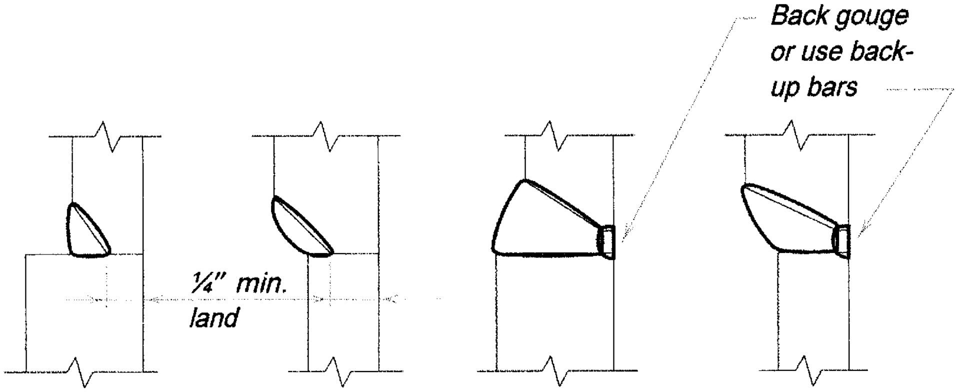
See the table below for minimum effective weld sizes for partial-penetration groove welds.

Partial Penetration Groove Width

[a]Thickness of Column Material T_u	Minimum Effective Weld Size E
[b]Over ½ to ¾, incl.	¼
Over ¾ to 1½, incl.	5⁄16
Over 1½ to 2¼, incl.	⅜
Over 2¼ to 6, incl.	½
Over 6	⅝

[a] Thickness of thinner part joined.

[b] For less than ½, use splice plates.



(a) Partial-joint-penetration groove welds

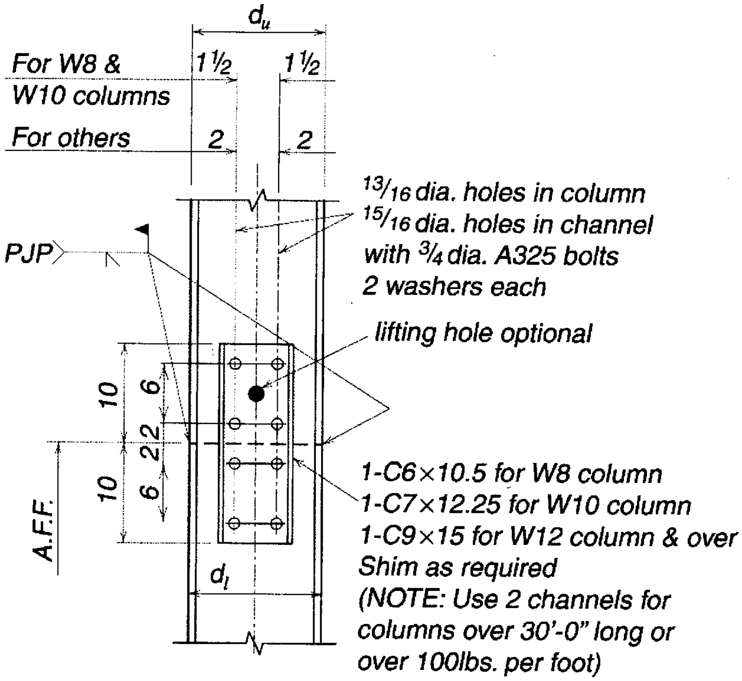
(b) Complete-joint-penetration groove welds

Table 14–3 (continued)

Typical Column Splices

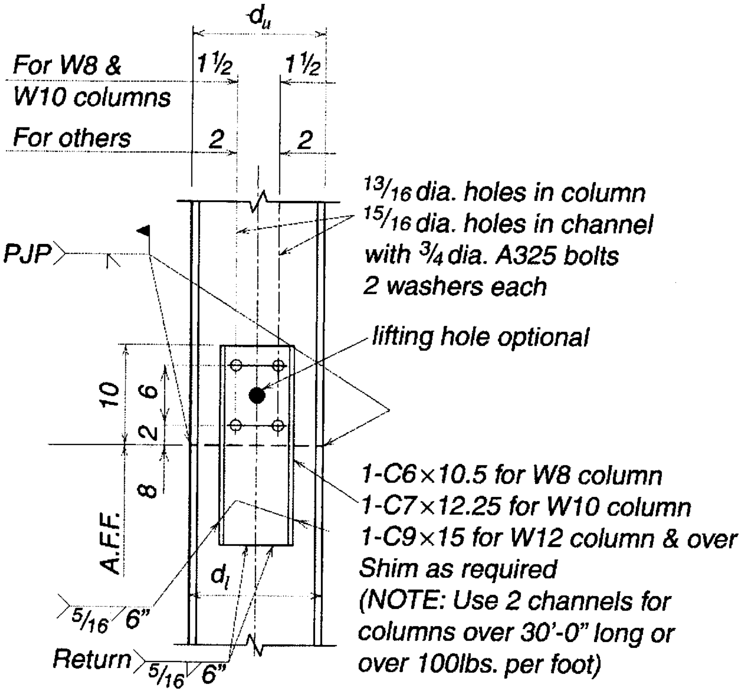
Case VIII:

Directly welded flange column splices between columns with depths d_u and d_l nominally the same.



CASE VIII A

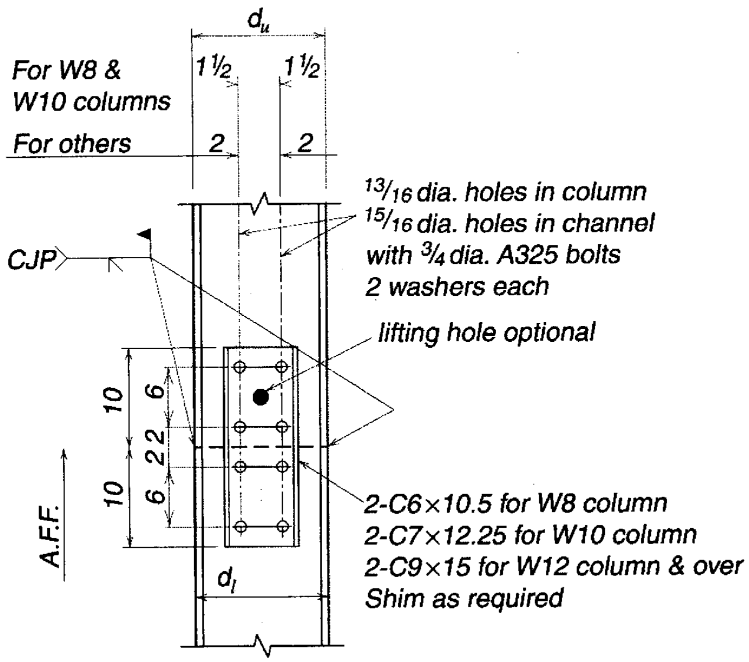
All-bolted web splice, partial-joint-penetration flange welds



CASE VIII B

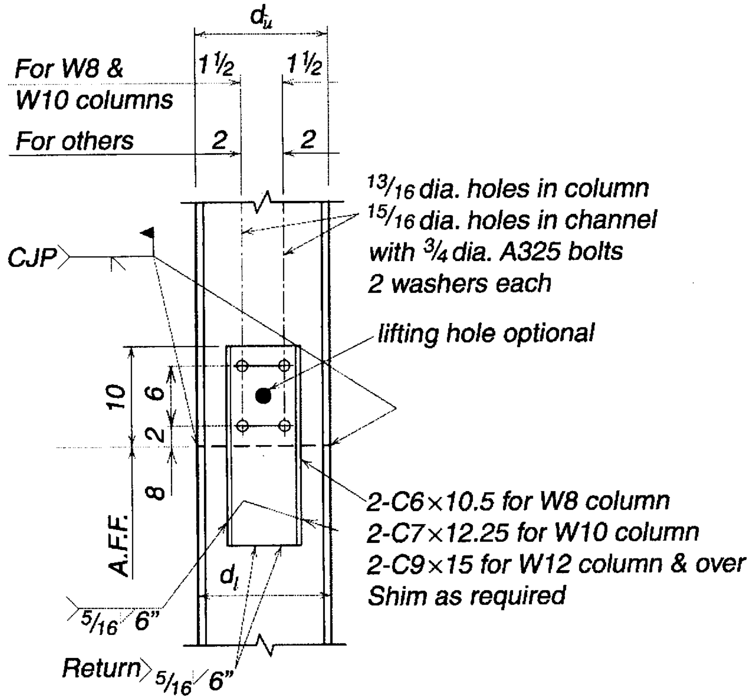
Combination bolted and welded web splice, partial-joint-penetration flange welds

Table 14–3 (continued)
Typical Column Splices
Case VIII:
Directly welded flange column splices between columns with depths d_u and d_l nominally the same.



CASE VIII C

All-bolted web splice, complete-joint-penetration flange welds



CASE VIII D

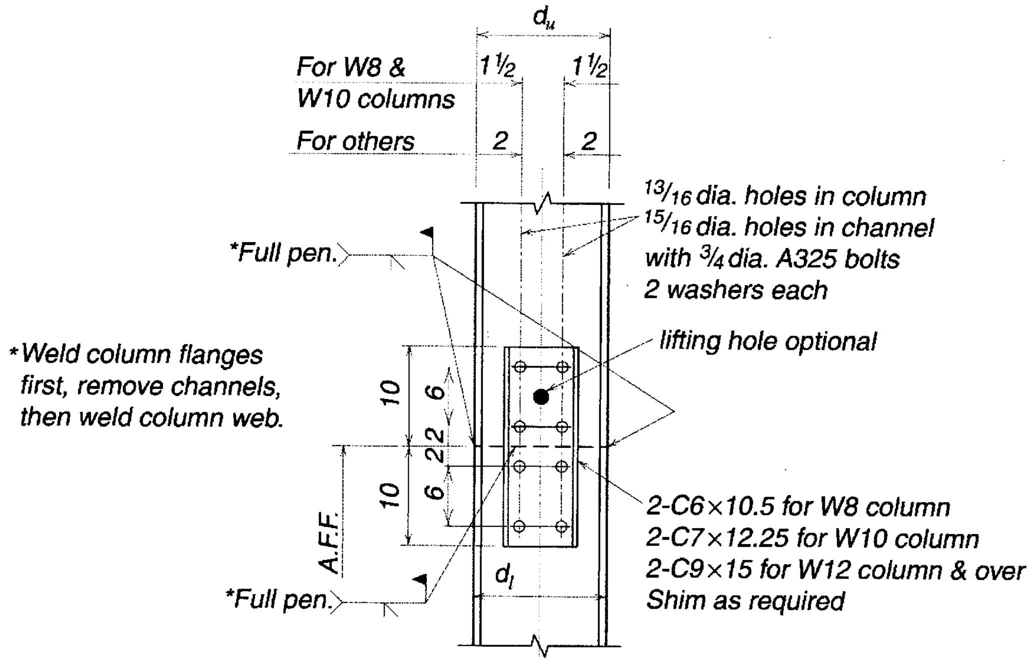
Combination bolted and welded web splice, complete-joint-penetration flange welds

Table 14–3 (continued)

Typical Column Splices

Case VIII:

Directly welded flange column splices between columns with depths d_u and d_l nominally the same.



CASE VIII E

web splice, complete-joint-penetration flange and web welds

Table 14–3 (continued)

Typical Column Splices

Case IX:

Butt-plated column splices between columns with depth d_u nominally two inches less than depth d_l.

Butt plate: Select a butt plate thickness of 1 ½-in. for W8 over W10 columns and 2 in. for all other combinations. Select butt plate width and length not less than w_l and d_l assuming the lower is the larger column shaft.

Weld: Select weld to upper column based on the thicker of t_{fu} and t_p. Select weld to lower column based on the thicker of t_{fl} and t_p. The edge preparation required by the groove weld is usually performed on the column shafts. However, special cases such as when the butt plate must be field welded to the lower column require special consideration.

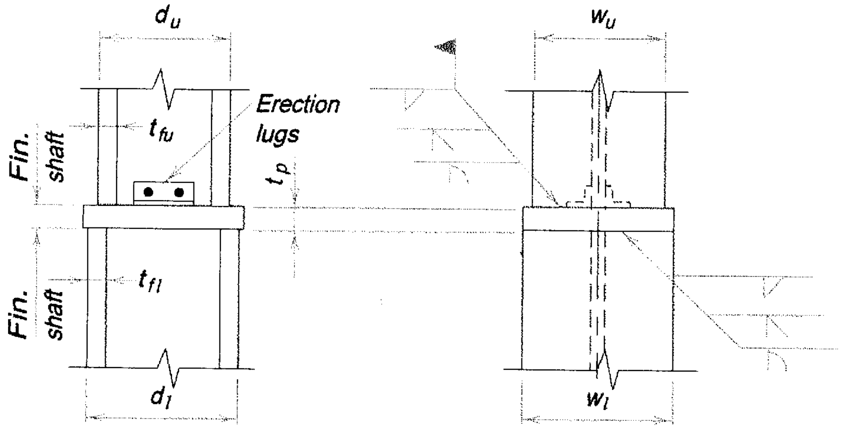
Erection: clip angles, such as those shown in the sketch below, help to locate and stabilize the upper column during the erection phase.

Table 14–3 (continued)

Typical Column Splices

Case IX:

Butt-plated column splices between columns with depth d_u nominally two inches less than depth d_l.



CASE IX

Table 14–3 (continued)

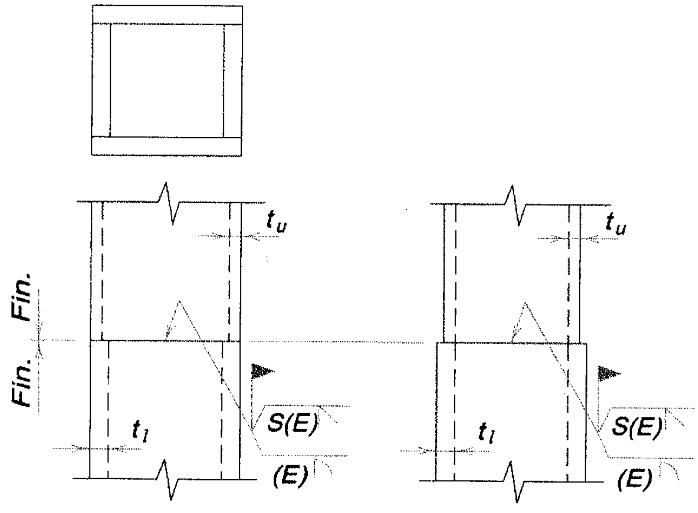
Typical Column Splices

Cases X, XI, XII

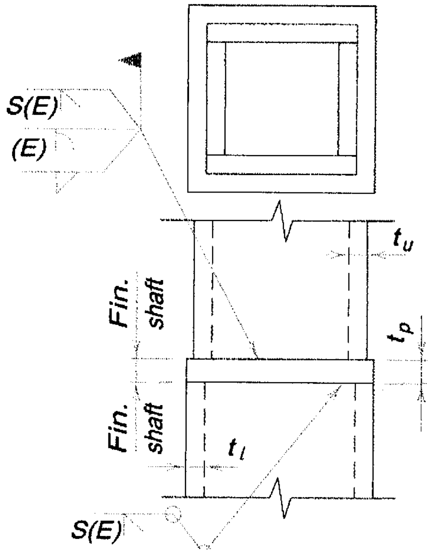
Special column splices.

Case X: Directly welded splice between tubular and/or box-shaped columns.	Welds may be either partial-joint- or complete-joint-penetration. The strength of partial-joint-penetration welds is a function of the column wall thickness and appropriate guidelines for minimum land width and effective weld size must be observed. This type of splice usually requires lifting and alignment devices. For lifting devices see Figure 11–21. For alignment devices see Figure 11–22.
Case XI: Butt-plated splices between tubular and/or box-shaped columns.	The butt-plate thickness is selected based on the AISC Specification. Welds may be either partial- or complete-penetration-groove welds, or, if adequate space is provided, fillet welds may be used. Weld strength is based on the thickness of connected material. See comments under Case X above regarding lifting and alignment devices.
Case XII: Butt-plated column splices between W-shape columns and tubular or box-shaped columns.	See comments under Case XI above.

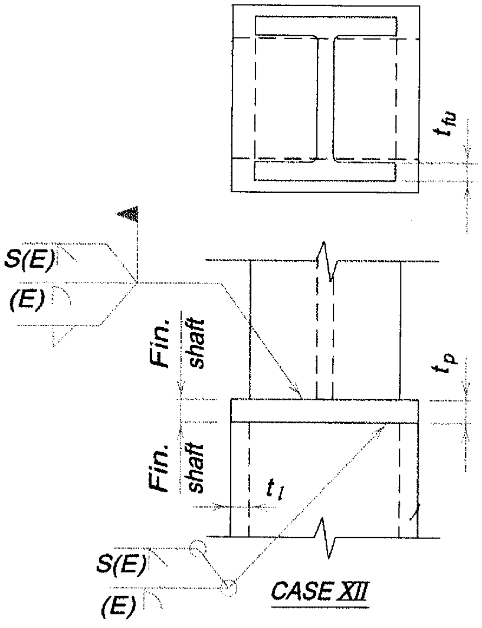
Table 14–3 (continued)
Typical Column Splices
Cases X, XI, XII
Special column splices.



CASE X



CASE XI



CASE XII

PART 15

DESIGN OF HANGER CONNECTIONS, BRACKET PLATES, AND CRANE-RAIL CONNECTIONS

SCOPE 15–2

HANGER CONNECTIONS 15–2

BRACKET PLATES 15–3

CRANE-RAIL CONNECTIONS 15–4

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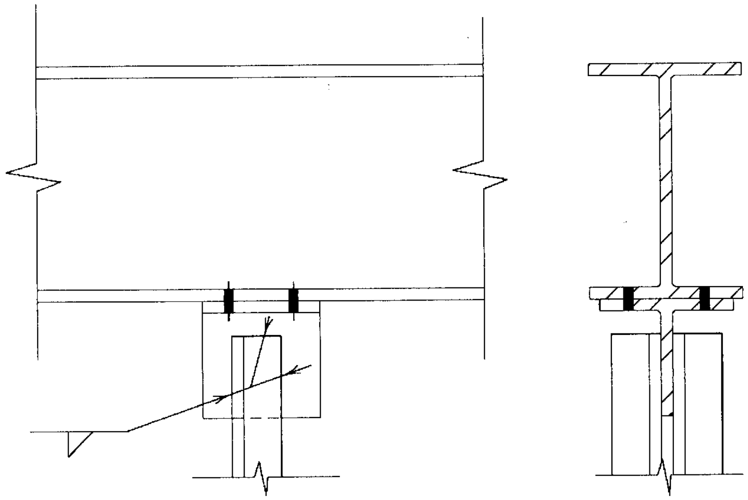
SCOPE

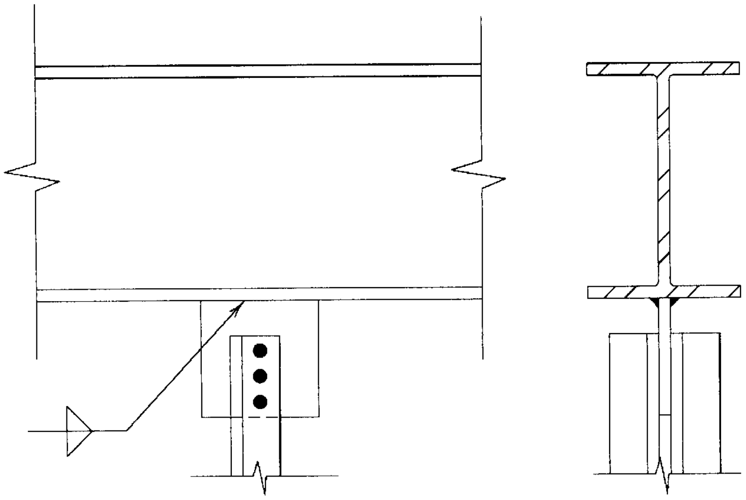
The specification requirements and other design considerations summarized in this Part apply to the design of hanger connections, bracket plates, and crane-rail connections. For the design of similar connections for HSS and pipe, see the AISC Specification Chapter K.

HANGER CONNECTIONS

Hanger connections, illustrated in Figure 15–1, are usually made with a plate, tee, angle, or pair of angles. The available strength of a hanger connection is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). In all cases, the available strength, ϕR_n or R_n/Ω, must exceed the required strength, R_u or R_a.



(a) Tee hanger



(b) Plate hanger

Figure 15–1. Typical hanger connections.

BRACKET PLATES

A bracket plate, illustrated in Figure 15–2, acts as a cantilevered beam. The available strength of a bracket plate is determined from the applicable limit states for the bolts (see Part 7), welds (see Part 8), and connecting elements (see Part 9). Additionally, special checks for flexural yielding, flexural rupture, and local buckling must considered, as follows.

For flexural yielding, the available strength, ϕM_n or M_n/Ω, of the bracket plate is determined with

$$M_n = F_y Z$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where Z is the gross plastic section modulus of the bracket plate. Additionally, triangular-shaped bracket plates should be checked for flexural yielding on the free edge (Salmon and Johnson, 1996). In lieu of a more detailed analysis, the load on the bracket plate can be limited by the available strength, ϕP_n or P_n/Ω, with

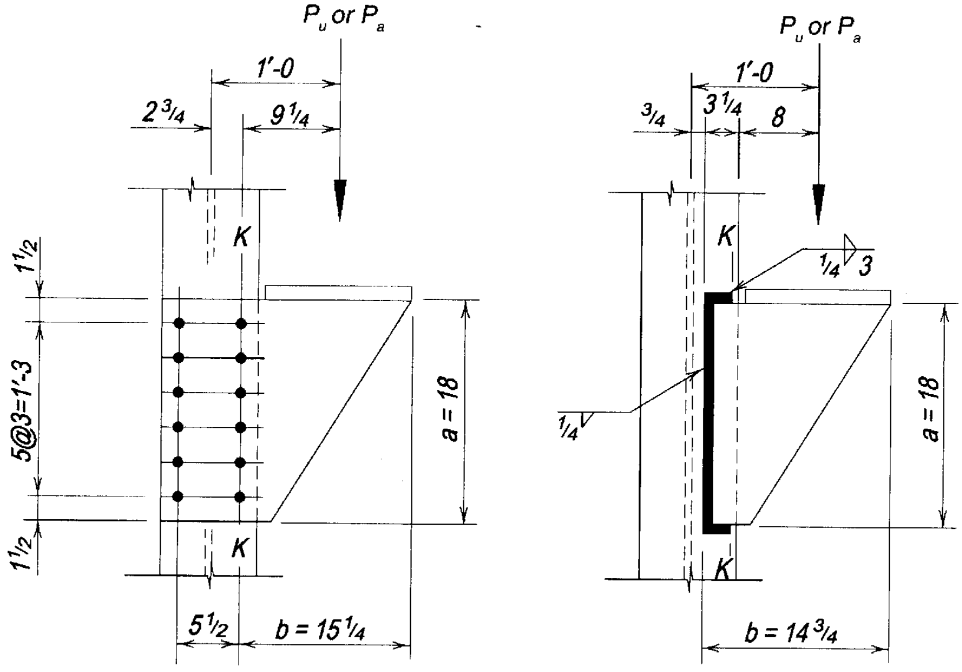
$$P_n = F_y zbt$$

$$\phi = 0.90 \qquad \Omega = 1.67$$

where

$$z = 1.39 - 2.2\left(\frac{b}{a}\right) + 1.27\left(\frac{b}{a}\right)^2 - 0.25\left(\frac{b}{a}\right)^3$$

b = width of bracket plate as shown in Figure 15–2, in.
a = depth of bracket plate as shown in Figure 15–2, in.
t = thickness of bracket plate, in.



(a) bolted (b) welded

Figure 15–2. Bracket-plate connections.

For flexural rupture, the available strength, ϕM_n or M_n/Ω, of the bracket plate is determined with

$$M_n = F_u Z_{net}$$

$$\phi = 0.75 \qquad \Omega = 2.00$$

where Z_{net} is the net plastic section modulus of the bracket plate. Values of Z_{net} are given in Table 15–2 for various bolt hole diameters and numbers of fasteners at 3-in. spacing.

Local buckling can be prevented in bracket plates if the following width-thickness ratios are satisfied, provided the centroid of the applied load is approximately 0.6*b* from the line of support (line K in Figure 15–2a) and lateral movement of the outstanding portion of the bracket plate is prevented (Salmon and Johnson, 1996)

$$\frac{b}{t} \leq \frac{250}{\sqrt{F_y}} \quad \text{for } 0.5 < \frac{b}{a} \leq 1.0$$

$$\frac{b}{t} \leq \frac{250}{\sqrt{F_y}}\left(\frac{b}{a}\right) \quad \text{for } 1.0 < \frac{b}{a} \leq 2.0.$$

CRANE-RAIL CONNECTIONS

Bolted Splices

It is desirable to use properly installed and maintained bolted splice bars in crane-rail connections rather than welded splice bars, which are frequently subject to failure in service.

Standard rail drilling and joint-bar punching, as furnished by manufacturers of light standard rails for track work, include round holes in rail ends and slotted holes in joint bars to receive standard oval-neck track bolts. Holes in rails are oversized and punching in joint bars is spaced to allow $^1/_{16}$-in. to $^1/_8$-in. clearance between rail ends (see manufacturers' catalogs for spacing and dimensions of holes and slots). Although this construction is satisfactory for track and light crane service, its use in general crane service may lead to high maintenance and joint failure. Welded splices are therefore preferable.

For best service in bolted splices, it is recommended that tight joints be required for all rails for crane service. This will require rail ends to be finished, and the special rail drilling and joint-bar punching tabulated below. Special rail drilling is accepted by some mills, or rails may be ordered blank for shop drilling. End finishing of standard rails can be done at the mill. However, light rails often must be end-finished in the shop or ground at the site prior to erection. In the crane rail range from 104 to 175 lbs per yard, rails and joint bars are manufactured to obtain a tight fit and no further special end finishing, drilling, or punching is required. Because of cumulative tolerance variations in holes, bolt diameters, and rail ends, a slight gap may sometimes occur. It may sometimes be necessary to ream holes through joined bar and rail to permit entry of bolts.

Joint bars for crane service are provided in various sections to match the rails. Joint bars for light and standard rails can be purchased blank for special shop punching to obtain tight joints. See manufacturer data for dimensions, material specifications, and the identification necessary to match the crane-rail section.

Joint-bar bolts, as distinguished from oval-neck track bolts, have straight shanks to the head and are manufactured to ASTM A449 specifications. Nuts are manufactured to ASTM A563 grade B specifications. Alternatively, ASTM A325 bolts and compatible ASTM A563 nuts can be used. Bolt assembly includes an alloy steel spring washer, furnished to American Railway Engineering and Maintenance of Way Association (AREMA) specifications. After installation, bolts should be retightened within 30 days and every three months thereafter.

Welded Splices

When welded splices are specified, consult the manufacturer for recommended rail-end preparation, welding procedure, and method of ordering. Although the joint continuity made possible by this method of splicing is desirable, the careful control required in all stages of the welding operation may be difficult to meet during crane-rail installation. Rails should not be attached to structural supports by welding. Rails with holes for joint bar bolts should not be used in making welded splices.

Hook Bolt Fastenings

Hook bolts (Figure 15–3) are used primarily with light rails when attached to beams that are too narrow for clamps. Rail adjustment to ±$^1/_2$ in. is inherent in the threaded shank. Hook bolts are paired alternately 3 to 4 in. apart, spaced at about 24 in. on center. The special rail drilling required must be done in the fabricator's shop. Hook bolts are not recommended for use with heavy-duty cycle cranes (Crane Manufacturers Association of America (CMAA) Classes, D, E, and F). It is generally recommended that hook bolts should not be used in runway systems that are longer than 500 ft because the bolts do not allow for longitudinal movement of the rail.

Rail Clip Fastenings

Rail clips are forged or cast devices that are shaped to match specific rail profiles. They are usually bolted to the runway girder flange with one bolt or are sometimes welded. Rail clips have been used satisfactorily with all classes of cranes. However, one drawback is that when

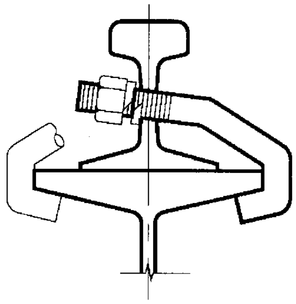


Figure 15–3. Hook bolts.

a single bolt is used, the clip can rotate in response to rail longitudinal movement. This clip rotation can cause cam action that might force the rail out of alignment. Because of this limitation, rail clips should only be used in crane systems subject to infrequent use, and for runways less than 500 ft in length.

Rail Clamp Fastenings

Rail clamps are a common method of attachment for heavy-duty cycle cranes. Rail clamps are detailed to provide two types: tight and floating (see Figure 15–4). Each clamp consists of two plates: an upper clamp plate and a lower filler plate. Dimensions shown are suggested. See manufacturers' catalogs for recommended gages, bolt sizes, and detail dimensions not shown.

The lower plate is flat and nominally matches the height of the toe of the rail flange. The upper plate covers the lower plate and extends over the top of the lower rail flange. In the tight clamp, the upper plate is detailed to fit tightly to the lower tail flange top, thus "clamping" it tightly in place when the fasteners are tightened. In the past, the tight clamp had been illustrated with the filler plates fitted tightly against the rail flange toe. This tight fit-up was rarely achieved in practice and is not considered to be necessary to achieve a tight type clamp. In the floating type clamp, the pieces are detailed to provide a clearance both alongside the rail flange toe and below the upper plate. The floating type does not, in reality, clamp the rail but merely holds the rail within the limits of the clamp clearances. High strength bolts are recommended for both clamp types. Both types should be spaced 3 ft or less apart.

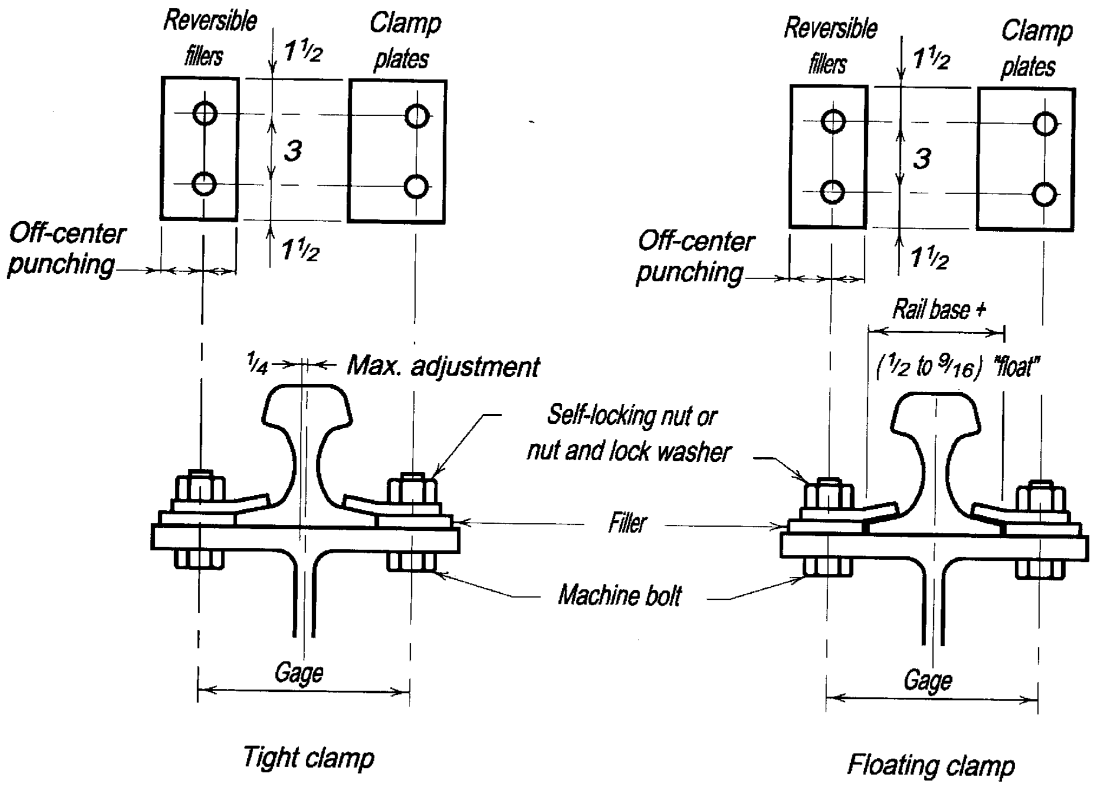


Figure 15–4. Rail clamps.

Patented Rail Clip Fastenings

Each manufacturer's literature presents in detail the desirable aspects of the various designs. In general, patented rail clips are easy to install due to their range of adjustment and provide both limitation of lateral movement and allowance for longitudinal movement. Patented rail clips should be considered as a viable alternative to conventional hook bolts, clips, or clamps. Because of their desirable characteristics, patented rail clips can be used without restriction except as limited by the specific manufacturer's recommendations. Installations using patented rail clips sometimes incorporate pads beneath the rail. When this is done, the lateral float of the rail should be limited as in the case of the tight rail clamps.

DESIGN TABLES

Table 15–1. Preliminary Hanger Connection Selection Table

Values are given for the available tensile strength per in. of fitting length in flexural yielding of a tee fitting flange with $F_y = 36$ ksi and $F_y = 50$ ksi. Table 15–1 can be used to select a trial fitting once the number and size of bolts required is known. The number of bolts required must be selected such that the available tensile strength of one bolt, ϕr_n or r_n/Ω, exceeds the required tensile force per bolt, r_{ut} or r_{at}.

In this table, it is assumed that equal critical moments exist at the face of the tee stem and at the bolt line. From AISC Specification Section F9, the available flexural yielding strength of the tee flange, $\phi_b M_n$ or M_n/Ω_b, is determined with

$$M_n = M_p = F_y Z$$

$$\phi_b = 0.90 \qquad \Omega_b = 1.67$$

In the above equation, the plastic section modulus Z_x per unit length of the tee flange is

$$Z_x = \frac{t^2}{4}$$

where t is the thickness of the angle or tee flange, in. Thus, for a unit length of the tee flange the available flexural strength, $\phi_b M_n$ or M_n/Ω_b, is determined with

$$M_n = \frac{F_y t^2}{4}$$

$$\phi_b = 0.90 \qquad \Omega_b = 1.67$$

and the tensile force on the fitting, $2r_{ut}$ or $2r_{at}$, must be such that

LRFD	ASD
$2r_{ut} \le \dfrac{0.9F_y t^2}{b}$	$2r_{at} \le \dfrac{F_y t^2}{1.67b}$

where b is the distance from bolt centerline to face of the tee stem or center of angle leg, in.

Table 15–2. Net Plastic Section Modulus, Z_{net}

Values of the net plastic section modulus Z_{net} are given in Table 15–2 for various hole diameters and numbers of fasteners spaced 3 in. on center, the usual spacing for these connections.

FORGED STEEL STRUCTURAL HARDWARE

Table 15–3. Dimensions and Weights of Clevises

Dimensions, weights, and available strengths of clevises are listed in Table 15–3.

Table 15–4. Clevis Numbers Compatible with Various Rods and Pins

Compatibility of clevises with various rods and pins is given in Table 15–4.

Table 15–5. Dimensions and Weights of Turnbuckles

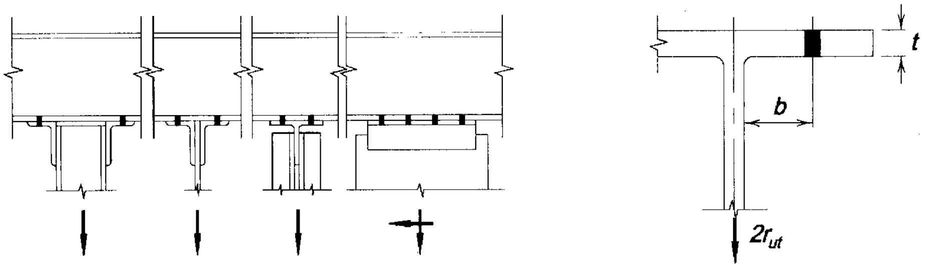
Dimensions, weights, and available strengths of turnbuckles are listed in Table 15–5.

PART 15 REFERENCES

Salmon, C.G. and J.E. Johnson, 1996, *Steel Structures: Design and Behavior*, 4th Edition, Harper Collins, New York, NY.

$F_y = 36$ ksi

Table 15–1a
Preliminary Hanger Connection Selection Table
Available tensile strength, kips per linear in., limited by flexural yielding of the flange



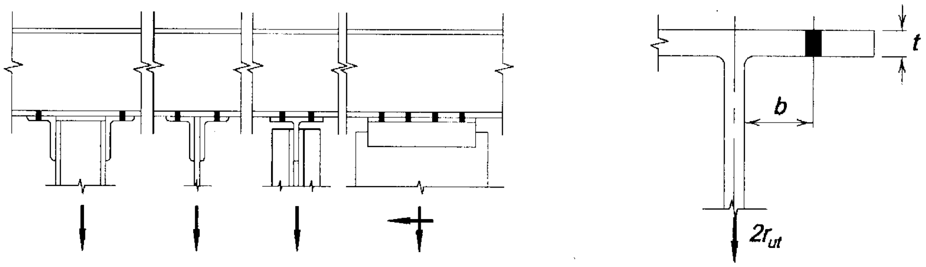
t, in.	*b*, in.									
	1		1 1/4		1 1/2		1 3/4		2	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
5/16	2.11	3.16	1.68	2.53	1.40	2.11	1.20	1.81	1.05	1.58
3/8	3.03	4.56	2.43	3.65	2.02	3.04	1.73	2.60	1.52	2.28
7/16	4.13	6.20	3.30	4.96	2.75	4.13	2.36	3.54	2.06	3.10
1/2	5.39	8.10	4.31	6.48	3.59	5.40	3.08	4.63	2.69	4.05
9/16	6.82	10.3	5.46	8.20	4.55	6.83	3.90	5.86	3.41	5.13
5/8	8.42	12.7	6.74	10.1	5.61	8.44	4.81	7.23	4.21	6.33
11/16	10.2	15.3	8.15	12.3	6.79	10.2	5.82	8.75	5.09	7.66
3/4	12.1	18.2	9.70	14.6	8.08	12.2	6.93	10.4	6.06	9.11
13/16	14.2	21.4	11.4	17.1	9.49	14.3	8.13	12.2	7.12	10.7
7/8	16.5	24.8	13.2	19.8	11.0	16.5	9.43	14.2	8.25	12.4
15/16	18.9	28.5	15.2	22.8	12.6	19.0	10.8	16.3	9.47	14.2
1	21.6	32.4	17.2	25.9	14.4	21.6	12.3	18.5	10.8	16.2
1 1/16	24.3	36.6	19.5	29.3	16.2	24.4	13.9	20.9	12.2	18.3
1 1/8	27.3	41.0	21.8	32.8	18.2	27.3	15.6	23.4	13.6	20.5
1 3/16	30.4	45.7	24.3	36.6	20.3	30.5	17.4	26.1	15.2	22.8
1 1/4	33.7	50.6	26.9	40.5	22.5	33.8	19.2	28.9	16.8	25.3
	2 1/4		2 1/2		2 3/4		3		3 1/4	
5/16	0.936	1.41	0.842	1.27	0.766	1.15	0.702	1.05	0.648	0.974
3/8	1.35	2.02	1.21	1.82	1.10	1.66	1.01	1.52	0.933	1.40
7/16	1.83	2.76	1.65	2.48	1.50	2.26	1.38	2.07	1.27	1.91
1/2	2.40	3.60	2.16	3.24	1.96	2.95	1.80	2.70	1.66	2.49
9/16	3.03	4.56	2.73	4.10	2.48	3.73	2.27	3.42	2.10	3.15
5/8	3.74	5.63	3.37	5.06	3.06	4.60	2.81	4.22	2.59	3.89
11/16	4.53	6.81	4.08	6.13	3.71	5.57	3.40	5.10	3.14	4.71
3/4	5.39	8.10	4.85	7.29	4.41	6.63	4.04	6.08	3.73	5.61
13/16	6.32	9.51	5.69	8.56	5.17	7.78	4.74	7.13	4.38	6.58
7/8	7.34	11.0	6.60	9.92	6.00	9.02	5.50	8.27	5.08	7.63
15/16	8.42	12.7	7.58	11.4	6.89	10.4	6.32	9.49	5.83	8.76
1	9.58	14.4	8.62	13.0	7.84	11.8	7.19	10.8	6.63	9.97
1 1/16	10.8	16.3	9.73	14.6	8.85	13.3	8.11	12.2	7.49	11.3
1 1/8	12.1	18.2	10.9	16.4	9.92	14.9	9.09	13.7	8.39	12.6
1 3/16	13.5	20.3	12.2	18.3	11.1	16.6	10.1	15.2	9.35	14.1
1 1/4	15.0	22.5	13.5	20.3	12.2	18.4	11.2	16.9	10.4	15.6

F_y = 50 ksi

Table 15–1b

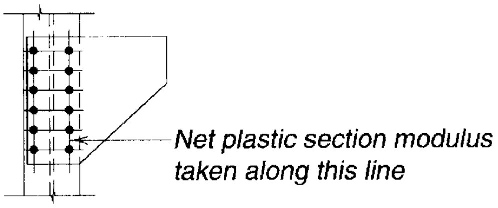
Preliminary Hanger Connection Selection Table

Available tensile strength, kips per linear in., limited by flexural yielding of the flange



t, in.	b, in.									
	1		1 1/4		1 1/2		1 3/4		2	
	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD	ASD	LRFD
5/16	2.92	4.39	2.34	3.52	1.95	2.93	1.67	2.51	1.46	2.20
3/8	4.21	6.33	3.37	5.06	2.81	4.22	2.41	3.62	2.11	3.16
7/16	5.73	8.61	4.58	6.89	3.82	5.74	3.27	4.92	2.87	4.31
1/2	7.49	11.3	5.99	9.00	4.99	7.50	4.28	6.43	3.74	5.63
9/16	9.47	14.2	7.58	11.4	6.32	9.49	5.41	8.14	4.74	7.12
5/8	11.7	17.6	9.36	14.1	7.80	11.7	6.68	10.0	5.85	8.79
11/16	14.2	21.3	11.3	17.0	9.43	14.2	8.09	12.2	7.08	10.6
3/4	16.8	25.3	13.5	20.3	11.2	16.9	9.62	14.5	8.42	12.7
13/16	19.8	29.7	15.8	23.8	13.2	19.8	11.3	17.0	9.88	14.9
7/8	22.9	34.5	18.3	27.6	15.3	23.0	13.1	19.7	11.5	17.2
15/16	26.3	39.6	21.1	31.6	17.5	26.4	15.0	22.6	13.2	19.8
1	29.9	45.0	24.0	36.0	20.0	30.0	17.1	25.7	15.0	22.5
1 1/16	33.8	50.8	27.0	40.6	22.5	33.9	19.3	29.0	16.9	25.4
1 1/8	37.9	57.0	30.3	45.6	25.3	38.0	21.7	32.5	18.9	28.5
1 3/16	42.2	63.5	33.8	50.8	28.1	42.3	24.1	36.3	21.1	31.7
1 1/4	46.8	70.3	37.4	56.3	31.2	46.9	26.7	40.2	23.4	35.2
	2 1/4		2 1/2		2 3/4		3		3 1/4	
5/16	1.30	1.95	1.17	1.76	1.06	1.60	0.975	1.46	0.900	1.35
3/8	1.87	2.81	1.68	2.53	1.53	2.30	1.40	2.11	1.30	1.95
7/16	2.55	3.83	2.29	3.45	2.08	3.13	1.91	2.87	1.76	2.65
1/2	3.33	5.00	2.99	4.50	2.72	4.09	2.50	3.75	2.30	3.46
9/16	4.21	6.33	3.79	5.70	3.44	5.18	3.16	4.75	2.91	4.38
5/8	5.20	7.81	4.68	7.03	4.25	6.39	3.90	5.86	3.60	5.41
11/16	6.29	9.45	5.66	8.51	5.15	7.73	4.72	7.09	4.35	6.54
3/4	7.49	11.3	6.74	10.1	6.12	9.20	5.61	8.44	5.18	7.79
13/16	8.78	13.2	7.91	11.9	7.19	10.8	6.59	9.90	6.08	9.14
7/8	10.2	15.3	9.17	13.8	8.34	12.5	7.64	11.5	7.05	10.6
15/16	11.7	17.6	10.5	15.8	9.57	14.4	8.77	13.2	8.10	12.2
1	13.3	20.0	12.0	18.0	10.9	16.4	9.98	15.0	9.21	13.8
1 1/16	15.0	22.6	13.5	20.3	12.3	18.5	11.3	16.9	10.4	15.6
1 1/8	16.8	25.3	15.2	22.8	13.8	20.7	12.6	19.0	11.7	17.5
1 3/16	18.8	28.2	16.9	25.4	15.4	23.1	14.1	21.2	13.0	19.5
1 1/4	20.8	31.3	18.7	28.1	17.0	25.6	15.6	23.4	14.4	21.6

Table 15–2
Net Plastic Section Modulus Z_{net}, in.^{3}



# Bolts in One Vertical Row n	Bracket Plate Depth d, in.	Nominal Bolt Diameter d_b, in.							
		$^3/_4$					$^7/_8$		
		Bracket Plate Thickness t, in.							
		$^1/_4$	$^3/_8$	$^1/_2$	$^5/_8$	$^3/_4$	$^3/_8$	$^1/_2$	$^5/_8$
2	**6**	1.59	2.39	3.19	3.98	4.78	2.25	3.00	3.75
3	**9**	3.70	5.55	7.40	9.26	11.1	5.25	7.00	8.75
4	**12**	6.38	9.56	12.8	15.9	19.1	9.00	12.0	15.0
5	**15**	10.1	15.1	20.2	25.2	30.2	14.3	19.0	23.8
6	**18**	14.3	21.5	28.7	35.9	43.0	20.3	27.0	33.8
7	**21**	19.6	29.5	39.3	49.1	58.9	27.8	37.0	46.3
8	**24**	25.5	38.3	51.0	63.8	76.5	36.0	48.0	60.0
9	**27**	32.4	48.6	64.8	81.0	97.2	45.8	61.0	76.3
10	**30**	39.8	59.8	79.7	99.6	120	56.3	75.0	93.8
12	**36**	57.4	86.1	115	143	172	81.0	108	135
14	**42**	78.1	117	156	195	234	110	147	184
16	**48**	102	153	204	255	306	144	192	240
18	**54**	129	194	258	323	387	182	243	304
20	**60**	159	239	319	398	478	225	300	375
22	**66**	193	289	386	482	579	272	363	454
24	**72**	230	344	459	574	689	324	432	540
26	**78**	269	404	539	673	808	380	507	634
28	**84**	312	469	625	781	937	441	588	735
30	**90**	359	538	717	896	1080	506	675	844
32	**96**	408	612	816	1020	1220	576	768	960
34	**102**	461	691	921	1150	1380	650	867	1080
36	**108**	516	775	1030	1290	1550	729	972	1220

Notes:

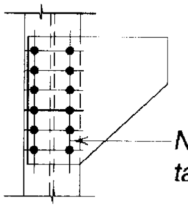
The area reduction per hole is assumed to be $d_h + {}^1/_{16}$ in.

Bolts spaced 3 in. vertically with $1^1/_2$ in. edge distance at top and bottom.

Interpolate for intermediate plate thicknesses.

Table 15–2 (continued)

Net Plastic Section Modulus Z_{net}, in.^{3}



Net plastic section modulus taken along this line

# Bolts in One Vertical Row n	Bracket Plate Depth d, in.	Nominal Bolt Diameter d_b, in.						
		$^7/_8$		1				
		Bracket Plate Thickness t, in.						
		$^3/_4$	$^7/_8$	$^1/_2$	$^5/_8$	$^3/_4$	$^7/_8$	1
2	**6**	4.50	5.25	2.81	3.52	4.22	4.92	5.63
3	**9**	10.5	12.3	6.59	8.24	9.89	11.5	13.2
4	**12**	18.0	21.0	11.3	14.1	16.9	19.7	22.5
5	**15**	28.5	33.3	17.8	22.3	26.8	31.2	35.7
6	**18**	40.5	47.3	25.3	31.6	38.0	44.3	50.6
7	**21**	55.5	64.8	34.7	43.4	52.1	60.8	69.4
8	**24**	72.0	84.0	45.0	56.3	67.5	78.8	90.0
9	**27**	91.5	107	57.2	71.5	85.8	100	114
10	**30**	113	131	70.3	87.9	105	123	141
12	**36**	162	189	101	127	152	177	203
14	**42**	221	257	138	172	207	241	276
16	**48**	288	336	180	225	270	315	360
18	**54**	365	425	228	285	342	399	456
20	**60**	450	525	281	352	422	492	563
22	**66**	545	635	340	425	510	596	681
24	**72**	648	756	405	506	608	709	810
26	**78**	761	887	475	594	713	832	951
28	**84**	882	1030	551	689	827	965	1100
30	**90**	1010	1180	633	791	949	1110	1270
32	**96**	1150	1340	720	900	1080	1260	1440
34	**102**	1300	1520	813	1020	1220	1420	1630
36	**108**	1460	1700	911	1140	1370	1590	1820

Notes:

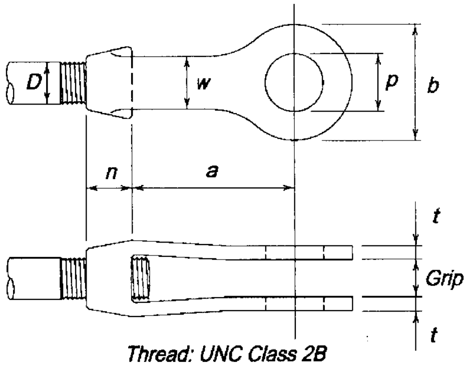
The area reduction per hole is assumed to be $d_h + {}^1/_{16}$ in.

Bolts spaced 3 in. vertically with $1^1/_2$ in. edge distance at top and bottom.

Interpolate for intermediate plate thicknesses.

Table 15–3

Dimensions and Weights of Clevises



Thread: UNC Class 2B

Grip = plate thickness + ¼ in.

Clevis Number	Dimensions, in.							Weight, pounds	Available Strength, kips	
	Max. *D*	Max. *p*	*b*	*n*	*a*	*w*	*t*		ASD	LRFD
2	$\frac{5}{8}$	$\frac{3}{4}$	$1\frac{7}{16}$	$\frac{5}{8}$	$3\frac{9}{16}$	$1\frac{1}{16}$	$\frac{5}{16}$ $(+\frac{1}{32}, -0)$	1	5.83	8.75
$2\frac{1}{2}$	$\frac{7}{8}$	$1\frac{1}{2}$	$2\frac{1}{2}$	1	4	$1\frac{1}{4}$	$\frac{5}{16}$ $(+\frac{1}{32}, -0)$	2.5	12.5	18.8
3	$1\frac{3}{8}$	$1\frac{3}{4}$	3	$1\frac{1}{4}$	$5\frac{1}{16}$	$1\frac{1}{2}$	$\frac{1}{2}$ $(+\frac{1}{16}, -\frac{1}{32})$	4	25.0	37.5
$3\frac{1}{2}$	$1\frac{1}{2}$	2	$3\frac{1}{2}$	$1\frac{1}{2}$	6	$1\frac{3}{4}$	$\frac{1}{2}$ $(+\frac{1}{16}, -\frac{1}{16})$	6	30.0	45.0
4	$1\frac{3}{4}$	$2\frac{1}{4}$	4	$1\frac{3}{4}$	$5\frac{15}{16}$	2	$\frac{1}{2}$ $(+\frac{1}{16}, -\frac{1}{16})$	9	35.0	52.5
5	$2\frac{1}{8}$	$2\frac{1}{2}$	5	$2\frac{1}{4}$	7	$2\frac{1}{2}$	$\frac{5}{8}$ $(+\frac{3}{32}, -0)$	16	62.5	93.8
6	$2\frac{1}{2}$	3	6	$2\frac{3}{4}$	8	3	$\frac{3}{4}$ $(+\frac{3}{32}, -0)$	26	90.0	135
7	3	$3\frac{3}{4}$	7	3	9	$3\frac{1}{2}$	$\frac{7}{8}$ $(+\frac{1}{8}, -\frac{1}{16})$	36	114	171
8	4	$4\frac{1}{4}$	8	4	$10\frac{1}{8}$	4	$1\frac{1}{2}$ $(+\frac{1}{8}, -\frac{1}{16})$	90	225	338

Notes:

Weights and dimensions of clevises are typical; products of all suppliers are essentially similar. User shall verify with the manufacturer that product meets available strength specifications above.

* Tabulated available strengths are based on $\phi = 0.5$, $\Omega = 3.0$. Strength at service load corresponds to a 3:1 safety factor using maximum pin diameter.

Table 15–4
Clevis Numbers Compatible with Various Rods and Pins

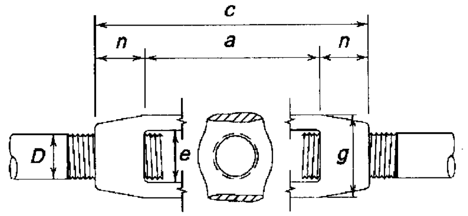
Dia. of Tap, in.	Diameter of Pin, in.																	
	1/2	5/8	3/4	7/8	1	1 1/4	1 1/2	1 3/4	2	2 1/4	2 1/2	2 3/4	3	3 1/4	3 1/2	3 3/4	4	4 1/4
3/8	2	2	2															
1/2	2	2	2															
5/8	2	2	2	2 1/2	2 1/2	2 1/2	2 1/2											
3/4			2 1/2	2 1/2	2 1/2	2 1/2	2 1/2											
7/8				2 1/2	2 1/2	2 1/2	2 1/2	3										
1					3	3	3	3										
1 1/8					3	3	3	3	3 1/2									
1 1/4					3	3	3	3	3 1/2									
1 3/8						3	3	3 1/2	3 1/2	4								
1 1/2						3 1/2	3 1/2	4	4	5								
1 5/8						4	4	4	5	5	5							
1 3/4							4	5	5	5	5							
1 7/8							5	5	5	5	5							
2							5	5	5	5	5	6	6					
2 1/8								5	5	6	6	6	6					
2 1/4									6	6	6	6	6	7	7			
2 3/8									6	6	6	6	7	7	7	7		
2 1/2									6	6	6	7	7	7	7	7		
2 5/8											7	7	7	7	7	8		
2 3/4											7	7	7	7	8	8		
2 7/8											7	8	8	8	8	8	8	8
3											7	8	8	8	8	8	8	8
3 1/8												8	8	8	8	8	8	8
3 1/4												8	8	8	8	8	8	8
3 3/8												8	8	8	8	8	8	8
3 1/2													8	8	8	8	8	8
3 5/8													8	8	8	8	8	
3 3/4													8	8	8	8	8	
3 7/8														8	8	8		
4														8	8			

Notes:

Tabular values assume that the net area of the clevis through the pin hole is greater than or equal to 125 percent of the net area of the rod, and is applicable to round rods without upset ends. For other net area ratios, the required clevis size may be calculated by referring to the dimensions tabulated in Table 15–3 and 7–18.

Table 15–5

Dimensions and Weights of Turnbuckles



Threads: UNC and 4UN Class 2B

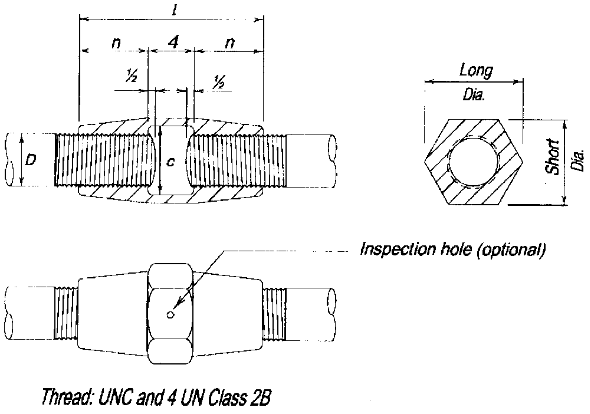
Diameter D, in.	Dimensions, in.					Weight (pounds) for Length a, in.						Available Strength, kips	
												ASD	LRFD
	a	n	c	e	g	6	9	12	18	24	26	R_n^*/Ω	ϕR_n^*
$^{3}/_{8}$	6	$^{9}/_{16}$	$7^{1}/_{8}$	$^{9}/_{16}$	$1^{1}/_{32}$	0.42						2.00	3.00
$^{1}/_{2}$	6	$^{25}/_{32}$	$7^{9}/_{16}$	$^{11}/_{16}$	$1^{5}/_{16}$	0.65	0.90	1.20				3.67	5.50
$^{5}/_{8}$	6	$^{15}/_{16}$	$7^{7}/_{8}$	$^{13}/_{16}$	$1^{1}/_{2}$	0.98	1.35	1.58	2.43			5.83	8.75
$^{3}/_{4}$	6	$1^{1}/_{16}$	$8^{1}/_{8}$	$^{15}/_{16}$	$1^{23}/_{32}$	1.45	1.84	2.35	3.06	4.25		8.67	13.0
$^{7}/_{8}$	6	$1^{5}/_{16}$	$8^{5}/_{8}$	$1^{3}/_{32}$	$1^{7}/_{8}$	1.85		3.02	4.20	5.43		12.0	18.0
1	6	$1^{7}/_{16}$	$8^{7}/_{8}$	$1^{9}/_{32}$	$2^{1}/_{32}$	2.60		4.02	4.40	6.85	10.0	15.5	23.3
$1^{1}/_{8}$	6	$1^{9}/_{16}$	$9^{1}/_{8}$	$1^{13}/_{32}$	$2^{9}/_{32}$	4.06		4.70	6.10			19.3	29.0
$1^{1}/_{4}$	6	$1^{9}/_{16}$	$9^{1}/_{8}$	$1^{9}/_{16}$	$2^{17}/_{32}$	4.00		6.49	7.13	11.3	13.1	25.3	38.0
$1^{3}/_{8}$	6	$1^{13}/_{16}$	$9^{5}/_{8}$	$1^{11}/_{16}$	$2^{3}/_{4}$	6.15						29.0	43.5
$1^{1}/_{2}$	6	$1^{7}/_{8}$	$9^{3}/_{4}$	$1^{27}/_{32}$	$3^{1}/_{32}$	6.15		9.70	9.13	16.8	19.4	35.0	52.5
$1^{5}/_{8}$	6	$2^{1}/_{2}$	11	$1^{31}/_{32}$	$3^{9}/_{32}$	9.80						40.9	61.3
$1^{3}/_{4}$	6	$2^{1}/_{2}$	11	$2^{1}/_{8}$	$3^{9}/_{16}$	9.80		15.3	16.0	19.5		47.2	70.8
$1^{7}/_{8}$	6	$2^{13}/_{16}$	$11^{5}/_{8}$	$2^{3}/_{8}$	4	14.0		15.3				62.0	93.0
2	6	$2^{13}/_{16}$	$11^{5}/_{8}$	$2^{3}/_{8}$	4	14.0		15.3		27.5		62.0	93.0
$2^{1}/_{4}$	6	$3^{5}/_{16}$	$12^{5}/_{8}$	$2^{11}/_{16}$	$4^{5}/_{8}$	19.6		30.9		43.5		80.0	120
$2^{1}/_{2}$	6	$3^{3}/_{4}$	$13^{1}/_{2}$	3	5	23.3		30.9		42.4		100	150
$2^{3}/_{4}$	6	$4^{3}/_{16}$	$14^{3}/_{8}$	$3^{1}/_{4}$	$5^{5}/_{8}$	31.5				54.0		125	188
3	6	$4^{5}/_{16}$	$14^{5}/_{8}$	$3^{5}/_{8}$	$6^{1}/_{8}$	39.5						161	242
$3^{1}/_{4}$	6	$5^{7}/_{16}$	$16^{7}/_{8}$	$3^{7}/_{8}$	$6^{3}/_{4}$	60.5		79.5				203	305
$3^{1}/_{2}$	6	$5^{7}/_{16}$	$16^{7}/_{8}$	$3^{7}/_{8}$	$6^{3}/_{4}$	60.5	70.0	79.5				203	305
$3^{3}/_{4}$	6	6	18	$4^{5}/_{8}$	$8^{1}/_{2}$	95.0						280	420
4	6	6	18	$4^{5}/_{8}$	$8^{1}/_{2}$	95.0						280	420
$4^{1}/_{4}$	9	$6^{3}/_{4}$	$22^{1}/_{2}$	$5^{1}/_{4}$	$9^{3}/_{4}$		152					390	585
$4^{1}/_{2}$	9	$6^{3}/_{4}$	$22^{1}/_{2}$	$5^{1}/_{4}$	$9^{3}/_{4}$		152					390	585
$4^{3}/_{4}$	9	$6^{3}/_{4}$	$22^{1}/_{2}$	$5^{1}/_{4}$	$9^{3}/_{4}$		152					390	585
5	9	$7^{1}/_{2}$	24	6	10		200					491	737

Notes:

Weights and dimensions of turnbuckles are typical; products of all suppliers are essentially similar. Users shall verify with the manufacturer that product meets strength specifications above.

* Tabulated available strengths are based on $\phi = 0.5$, $\Omega = 3.0$.

Table 15–6
Dimensions and Weights of Sleeve Nuts



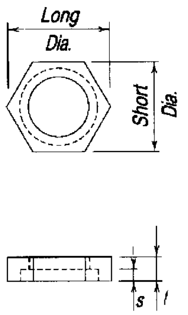
Thread: UNC and 4 UN Class 2B

Screw Dia. *D*, in.	Dimensions, in.					Weight, pounds
	Short Dia.	Long Dia.	Length *l*	Nut *n*	Clear *c*	
3/8	11/16	25/32	4	–	–	0.27
7/16	25/32	7/8	4	–	–	0.34
1/2	7/8	1	4	–	–	0.43
9/16	15/16	1 1/16	5	–	–	0.64
5/8	1 1/16	1 7/32	5	–	–	0.93
3/4	1 1/4	1 7/16	5	–	–	1.12
7/8	1 7/16	1 5/8	7	1 7/16	1	1.75
1	1 5/8	1 13/16	7	1 7/16	1 1/8	2.46
1 1/8	1 13/16	2 1/16	7 1/2	1 5/8	1 1/4	3.10
1 1/4	2	2 1/4	7 1/2	1 5/8	1 3/8	4.04
1 3/8	2 3/16	2 1/2	8	1 7/8	1 1/2	4.97
1 1/2	2 3/8	2 11/16	8	1 7/8	1 5/8	6.16
1 5/8	2 9/16	2 15/16	8 1/2	2 1/16	1 3/4	7.36
1 3/4	2 3/4	3 1/8	8 1/2	2 1/16	1 7/8	8.87
1 7/8	2 15/16	3 5/16	9	2 5/16	2	10.4
2	3 1/8	3 1/2	9	2 5/16	2 1/8	12.2
2 1/4	3 1/2	3 15/16	9 1/2	2 1/2	2 3/8	16.2
2 1/2	3 7/8	4 3/8	10	2 3/4	2 5/8	21.1
2 3/4	4 1/4	4 13/16	10 1/2	2 15/16	2 7/8	26.7
3	4 5/8	5 1/4	11	3 3/16	3 1/8	33.2
3 1/4	5	5 5/8	11 1/2	3 3/8	3 3/8	40.6
3 1/2	5 3/8	6	12	3 5/8	3 5/8	49.1
3 3/4	5 3/4	6 3/8	12 1/2	3 13/16	3 7/8	58.6
4	6 1/8	6 7/8	13	4 1/16	4 1/8	69.2
4 1/4	6 1/2	7 1/2	13 1/2	4 3/4	4 3/8	75.0
4 1/2	6 7/8	7 15/16	14	5	4 3/4	90.0
4 3/4	7 1/4	8 3/8	14 1/2	5 1/4	5	98.0
5	7 5/8	8 7/8	15	5 1/2	5 1/4	110
5 1/4	8	9 1/4	15 1/2	5 3/4	5 1/2	122
5 1/2	8 3/8	9 3/4	16	6	5 3/4	142
5 3/4	8 3/4	10 1/8	16 1/2	6 1/4	6	157
6	9 1/8	10 5/8	17	6 1/2	6 1/4	176

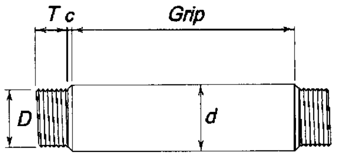
Notes:
Weights and dimensions of sleeve nuts are typical; products of all suppliers are essentially similar. User shall verify with the manufacturer that strengths of sleeve nut are greater than the corresponding connecting rod when the same material is used.

Table 15–7

Dimensions and Weights of Recessed-Pin Nuts

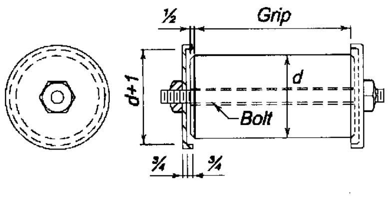


Material: Steel



Thread: 6 UN Class 2A/2B

Pin Dia. d, in.	Pin Dimensions, in			Nut Dimensions, in.					Weight, pounds
	Thread		c	Thick-ness t	Diameter		Recess		
	D	T			Short Dia.	Long Dia.	Rough Dia.	s	
2, $2\frac{1}{4}$	$1\frac{1}{2}$	1	$\frac{1}{8}$	$\frac{7}{8}$	3	$3\frac{3}{8}$	$2\frac{5}{8}$	$\frac{1}{4}$	1
$2\frac{1}{2}$, $2\frac{3}{4}$	2	$1\frac{1}{8}$	$\frac{1}{8}$	1	$3\frac{5}{8}$	$4\frac{1}{8}$	$3\frac{1}{8}$	$\frac{1}{4}$	2
3, $3\frac{1}{4}$, $3\frac{1}{2}$	$2\frac{1}{2}$	$1\frac{1}{4}$	$\frac{1}{8}$	$1\frac{1}{8}$	$4\frac{3}{8}$	5	$3\frac{7}{8}$	$\frac{3}{8}$	3
$3\frac{3}{4}$, 4	3	$1\frac{3}{8}$	$\frac{1}{4}$	$1\frac{1}{4}$	$4\frac{7}{8}$	$5\frac{5}{8}$	$4\frac{3}{8}$	$\frac{3}{8}$	4
$4\frac{1}{4}$, $4\frac{1}{2}$, $4\frac{3}{4}$	$3\frac{1}{2}$	$1\frac{1}{2}$	$\frac{1}{4}$	$1\frac{3}{8}$	$5\frac{3}{4}$	$6\frac{5}{8}$	$5\frac{1}{4}$	$\frac{1}{2}$	5
5, $5\frac{1}{4}$	4	$1\frac{5}{8}$	$\frac{1}{4}$	$1\frac{1}{2}$	$6\frac{1}{4}$	$7\frac{1}{4}$	$5\frac{3}{4}$	$\frac{1}{2}$	6
$5\frac{1}{2}$, $5\frac{3}{4}$, 6	$4\frac{1}{2}$	$1\frac{3}{4}$	$\frac{1}{4}$	$1\frac{5}{8}$	7	$8\frac{1}{8}$	$6\frac{1}{2}$	$\frac{5}{8}$	8
$6\frac{1}{4}$, $6\frac{1}{2}$	5	$1\frac{7}{8}$	$\frac{3}{8}$	$1\frac{3}{4}$	$7\frac{5}{8}$	$8\frac{7}{8}$	7	$\frac{5}{8}$	10
$6\frac{3}{4}$, 7	$5\frac{1}{2}$	2	$\frac{3}{8}$	$1\frac{7}{8}$	$8\frac{1}{8}$	$9\frac{3}{8}$	$7\frac{1}{2}$	$\frac{3}{4}$	12
$7\frac{1}{4}$, $7\frac{1}{2}$	$5\frac{1}{2}$	2	$\frac{3}{8}$	$1\frac{7}{8}$	$8\frac{5}{8}$	10	8	$\frac{3}{4}$	14
$7\frac{3}{4}$, 8, $8\frac{1}{4}$	6	$2\frac{1}{4}$	$\frac{3}{8}$	$2\frac{1}{8}$	$9\frac{3}{8}$	$10\frac{7}{8}$	$8\frac{3}{4}$	$\frac{3}{4}$	19
$8\frac{1}{2}$, $8\frac{3}{4}$, 9	6	$2\frac{1}{4}$	$\frac{3}{8}$	$2\frac{1}{8}$	$10\frac{1}{4}$	$11\frac{7}{8}$	$9\frac{5}{8}$	$\frac{3}{4}$	24
$9\frac{1}{4}$, $9\frac{1}{2}$	6	$2\frac{3}{8}$	$\frac{3}{8}$	$2\frac{1}{4}$	$11\frac{1}{4}$	13	$10\frac{5}{8}$	$\frac{3}{4}$	32
$9\frac{3}{4}$, 10	6	$2\frac{3}{8}$	$\frac{3}{8}$	$2\frac{1}{4}$	$11\frac{1}{4}$	13	$10\frac{5}{8}$	$\frac{3}{4}$	32



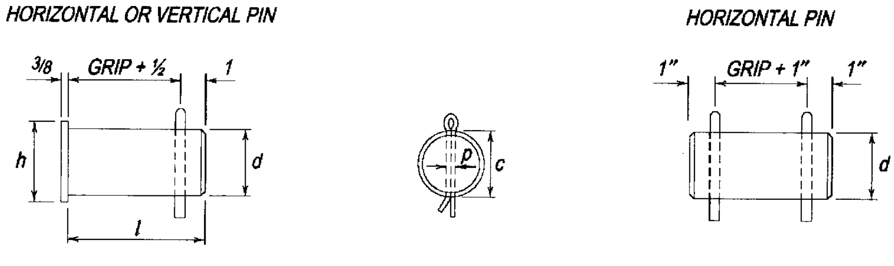
Typical Pin Cap Detail for Pins over 10 in. in dia.
Dimensions shown are approximate

Notes:

Although nuts may be used on all sizes of pins as shown above, a detail similar to that shown at the left is preferable for pin diameters over 10 in. In this detail, the pin is held in place by a recessed cap at each end and secured by a bolt passing completely through the caps and pin. Suitable provisions must be made for attaching pilots and driving nuts.

Table 15–8

Dimensions and Weights of Cotter Pins



l = Length of pin, in.

Pin Diameter *d*, in.	Pins with Heads		Cotter		
	Head Diameter *h*, in.	Weight of One pounds	Length *c*, in.	Diameter *p*, in.	Weight per 100, pounds
1 1/4	1 1/2	$0.19 + 0.35l$	7/8	1/4	2.64
1 1/2	1 3/4	$0.26 + 0.50l$	1	1/4	3.10
1 3/4	2	$0.33 + 0.68l$	1 1/8	1/4	3.50
2	2 3/8	$0.47 + 0.89l$	1 1/4	3/8	9.00
2 1/4	2 5/8	$0.58 + 1.13l$	1 3/8	3/8	9.40
2 1/2	2 7/8	$0.70 + 1.39l$	1 1/2	3/8	10.9
2 3/4	3 1/8	$0.82 + 1.68l$	1 5/8	3/8	11.4
3	3 1/2	$1.02 + 2.00l$	1 3/4	1/2	28.5
3 1/4	3 3/4	$1.17 + 2.35l$	1 7/8	1/2	28.5
3 1/2	4	$1.34 + 2.73l$	1 7/8	1/2	33.8
3 3/4	4 1/4	$1.51 + 3.13l$	2 1/4	1/2	33.8

ANSI/AISC 360-05
An American National Standard

Specification for Structural Steel Buildings

March 9, 2005

Supersedes the *Load and Resistance Factor Design Specification for Structural Steel Buildings* dated December 27, 1999, the *Specification for Structural Steel Buldings—Allowable Stress Design and Plastic Design* dated June 1, 1989, including Supplement No. 1, the *Specification for Allowable Stress Design of Single-Angle Members* dated June 1, 1989, the *Load and Resistance Factor Design Specification for Single-Angle Members* dated November 10, 2000, and the *Load and Resistance Factor Design Specification for the Design of Steel Hollow Structural Sections* dated November 10, 2000, and all previous versions of these specifications.

Approved by the AISC Committee on Specifications and issued by the AISC Board of Directors

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Printed in the United States of America

DEDICATION

Professor Lynn S. Beedle

This edition of the AISC Specification is dedicated to the memory of Dr. Lynn S. Beedle, University Distinguished Professor at Lehigh University. Dr. Beedle served as a faculty member at Lehigh University for 41 years and won a very large number of professional and educational awards, including the 1973 T.R. Higgins Award and the 2003 Geerhard Haaijer Award from AISC. He was a major contributor to several editions of the AISC Specification and a long-time member of the AISC Committee on Specifications. He was instrumental in the development of plastic design methodologies and its implementation into the AISC Specification. He was Director of the Structural Stability Research Council for 25 years, and in that role fostered understanding of various stability problems and helped develop rational design provisions, many of which were adopted in the AISC Specifications. In 1969, he founded the Council on Tall Buildings and Urban Habitat and succeeded in bringing together the disciplines of architecture, structural engineering, construction, environment, sociology and politics, which underlie every major tall building project. He was actively involved in this effort until his death in late 2003 at the age of 85. His contributions to the design and construction of steel buildings will long be remembered by AISC, the steel industry and the structural engineering profession worldwide.

For a more complete discussion of Dr. Beedle's life and accomplishments, see *Catalyst for Skyscraper Revolution: Lynn S. Beedle—A Legend in his Lifetime* by Mir Ali, published by the Council on Tall Buildings and Urban Habitat (2004).

PREFACE

(This Preface is not part of ANSI/AISC 360-05, *Specification for Structural Steel Buildings*, but is included for informational purposes only.)

This Specification has been based upon past successful usage, advances in the state of knowledge, and changes in design practice. The 2005 American Institute of Steel Construction's *Specification for Structural Steel Buildings* for the first time provides an integrated treatment of Allowable Stress Design (ASD) and Load and Resistance Factor Design (LRFD), and thus combines and replaces earlier Specifications that treated the two design methods separately. As indicated in Chapter B of the Specification, designs can be made according to either ASD or LRFD provisions.

This Specification has been developed as a consensus document to provide a uniform practice in the design of steel-framed buildings and other structures. The intention is to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems, which occur in the full range of structural design.

This Specification is the result of the consensus deliberations of a committee of structural engineers with wide experience and high professional standing, representing a wide geographical distribution throughout the United States. The committee includes approximately equal numbers of engineers in private practice and code agencies, engineers involved in research and teaching, and engineers employed by steel fabricating and producing companies. The contributions and assistance of more than 50 additional professional volunteers working in ten task committees are also hereby acknowledged.

The Symbols, Glossary and Appendices to this Specification are an integral part of the Specification. A non-mandatory Commentary has been prepared to provide background for the Specification provisions and the user is encouraged to consult it. Additionally, non-mandatory User Notes are interspersed throughout the Specification to provide concise and practical guidance in the application of the provisions.

The reader is cautioned that professional judgment must be exercised when data or recommendations in the Specification are applied, as described more fully in the disclaimer notice preceding this Preface.

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SYMBOLS

The section or table number in the right-hand column refers to where the symbol is first used.

Symbol	Definition	Section
A	Column cross-sectional area, in.^{2} (mm^2)	J10.6
A	Total cross-sectional area of member, in.^{2} (mm^2)	E7.2
A_B	Loaded area of concrete, in.^{2} (mm^2)	I2.1
A_{BM}	Cross-sectional area of the base metal, in.^{2} (mm^2)	J2.4
A_b	Nominal unthreaded body area of bolt or threaded part, in.^{2} (mm^2)	J3.6
A_{bi}	Cross-sectional area of the overlapping branch, in.^{2} (mm^2)	K2.3
A_{bj}	Cross-sectional area of the overlapped branch, in.^{2} (mm^2)	K2.3
A_c	Area of concrete, in.^{2} (mm^2)	I2.1
A_c	Area of concrete slab within effective width, in.^{2} (mm^2)	I3.2
A_D	Area of an upset rod based on the major thread diameter, in.^{2} (mm^2)	Table J3.2
A_e	Effective net area, in.^{2} (mm^2)	D2
A_{eff}	Summation of the effective areas of the cross section based on the reduced effective width, b_e, in.^{2} (mm^2)	E7.2
A_{fc}	Area of compression flange	G3.1
A_{fg}	Gross tension flange area, in.^{2} (mm^2)	F13.1
A_{fn}	Net tension flange area, in.^{2} (mm^2)	F13.1
A_{ft}	Area of tension flange, in.^{2} (mm^2)	G3.1
A_g	Gross area of member, in.^{2} (mm^2)	B3.13
A_g	Gross area of section based on design wall thickness, in.^{2} (mm^2)	G6
A_g	Gross area of composite member, in.^{2} (mm^2)	I2.1
A_g	Chord gross area, in.^{2} (mm^2)	K2.2
A_{gv}	Gross area subject to shear, in.^{2} (mm^2)	J4.3
A_n	Net area of member, in.^{2} (mm^2)	B3.13
A_{nt}	Net area subject to tension, in.^{2} (mm^2)	J4.3
A_{nv}	Net area subject to shear, in.^{2} (mm^2)	J4.2
A_{pb}	Projected bearing area, in.^{2} (mm^2)	J7
A_r	Area of adequately developed longitudinal reinforcing steel within the effective width of the concrete slab, in.^{2} (mm^2)	I3.2
A_s	Area of steel cross section, in.^{2} (mm^2)	I2.1
A_{sc}	Cross-sectional area of stud shear connector, in.^{2} (mm^2)	I2.1
A_{sf}	Shear area on the failure path, in.^{2} (mm^2)	D5.1
A_{sr}	Area of continuous reinforcing bars, in.^{2} (mm^2)	I2.1
A_{st}	Stiffener area, in.^{2} (mm^2)	G3.3
A_t	Net tensile area, in.^{2} (mm^2)	App. 3.4
A_w	Web area, the overall depth times the web thickness, dt_w, in.^{2} (mm^2)	G2.1

A_w	Effective area of the weld, in.^{2} (mm^2)	J2.4
A_{wi}	Effective area of weld throat of any ith weld element, in.^{2} (mm^2)	J2.4
A_1	Area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)	J8
A_2	Maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)	J8
B	Overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, in. (mm)	Table D3.1
B	Overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, in. (mm)	K3.1
B	Factor for lateral-torsional buckling in tees and double angles	F9.2
B_b	Overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, in. (mm)	K3.1
B_{bi}	Overall branch width of the overlapping branch	K2.3
B_{bj}	Overall branch width of the overlapped branch	K2.3
B_p	Width of plate, measure 90 degrees to the plane of the connection, in. (mm)	K1.1
B_p	Width of plate, transverse to the axis of the main member, in. (mm)	K2.3
B_1, B_2	Factors used in determining M_u for combined bending and axial forces when first-order analysis is employed	C2.1
C	HSS torsional constant	H3.1
C_b	Lateral-torsional buckling modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced	F1
C_d	Coefficient relating relative brace stiffness and curvature	App. 6.3.1
C_f	Constant based on stress category, given in Table A-3.1	App. 3.3
C_m	Coefficient assuming no lateral translation of the frame	C2.1
C_p	Ponding flexibility coefficient for primary member in a flat roof	App. 2.1
C_r	Coefficient for web sidesway buckling	J10.4
C_s	Ponding flexibility coefficient for secondary member in a flat roof	App. 2.1
C_v	Web shear coefficient	G2.1
C_w	Warping constant, in.^{6} (mm^6)	E4
D	Nominal dead load	App. 2.2
D	Outside diameter of round HSS member, in. (mm)	Table B4.1
D	Outside diameter, in. (mm)	E7.2
D	Outside diameter of round HSS main member, in. (mm)	K2.1
D	Chord diameter, in. (mm)	K2.2
D_b	Outside diameter of round HSS branch member, in. (mm)	K2.1
D_s	Factor used in Equation G3-3, dependent on the type of transverse stiffeners used in a plate girder	G3.3
D_u	In slip-critical connections, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension	J3.8
E	Modulus of elasticity of steel $= 29{,}000$ ksi (200 000 MPa)	Table B4.1
E_c	Modulus of elasticity of concrete $= w_c^{1.5}\sqrt{f_c'}$, ksi ($0.043 w_c^{1.5}\sqrt{f_c'}$, MPa)	I2.1

E_{cm}	Modulus of elasticity of concrete at elevated temperature, ksi (MPa)	App. 4.2.3
EI_{eff}	Effective stiffness of composite section, kip-in.^{2} (N-mm^2)	I2.1
E_m	Modulus of elasticity of steel at elevated temperature, ksi (MPa)	App. 4.2.3
E_s	Modulus of elasticity of steel = 29,000 ksi (200 000 MPa)	I2.1
F_a	Available axial stress at the point of consideration, ksi (MPa)	H2
F_{BM}	Nominal strength of the base metal per unit area, ksi (MPa)	J2.4
F_{bw}	Available flexural stress at the point of consideration about the major axis, ksi (MPa)	H2
F_{bz}	Available flexural stress at the point of consideration about the minor axis, ksi (MPa)	H2
F_c	Available stress, ksi (MPa)	K2.2
F_{cr}	Critical stress, ksi (MPa)	E3
F_{cr}	Buckling stress for the section as determined by analysis, ksi (MPa)	F12.2
F_{cry}	Critical stress about the minor axis, ksi (MPa)	E4
F_{crz}	Critical torsional buckling stress, ksi (MPa)	E4
F_e	Elastic critical buckling stress, ksi (MPa)	C1.3
F_{ex}	Elastic flexural buckling stress about the major axis, ksi (MPa)	E4
F_{EXX}	Electrode classification number, ksi (MPa)	J2.4
F_{ey}	Elastic flexural buckling stress about the minor axis, ksi (MPa)	E4
F_{ez}	Elastic torsional buckling stress, ksi (MPa)	E4
F_L	A calculated stress used in the calculation of nominal flexural strength, ksi (MPa)	Table B4.1
F_n	Nominal torsional strength	H3.3
F_n	Nominal tensile stress F_{nt}, or shear stress, F_{nv}, from Table J3.2, ksi (MPa)	J3.6
F_{nt}	Nominal tensile stress from Table J3.2, ksi (MPa)	J3.7
F'_{nt}	Nominal tensile stress modified to include the effects of shearing stress, ksi (MPa)	J3.7
F_{nv}	Nominal shear stress from Table J3.2, ksi (MPa)	J3.7
F_{SR}	Design stress range, ksi (MPa)	App. 3.3
F_{TH}	Threshold fatigue stress range, maximum stress range for indefinite design life from Table A-3.1, ksi (MPa)	App. 3.1
F_u	Specified minimum tensile strength of the type of steel being used, ksi (MPa)	D2
F_u	Specified minimum tensile strength of a stud shear connector, ksi (MPa)	I2.1
F_u	Specified minimum tensile strength of the connected material, ksi (MPa)	J3.10
F_u	Specified minimum tensile strength of HSS material, ksi (MPa)	K1.1
F_{um}	Specified minimum tensile strength of the type of steel being used at elevated temperature, ksi (MPa)	App. 4.2
F_w	Nominal strength of the weld metal per unit area, ksi (MPa)	J2.4
F_{wi}	Nominal stress in any ith weld element, ksi (MPa)	J2.4

F_{wix}	x component of stress F_{wi}, ksi (MPa)	J2.4
F_{wiy}	y component of stress F_{wi}, ksi (MPa)	J2.4
F_y	Specified minimum yield stress of the type of steel being used, ksi (MPa). As used in this Specification, "yield stress" denotes either the specified minimum yield point (for those steels that have a yield point) or specified yield strength (for those steels that do not have a yield point)	Table B4.1
F_y	Specified minimum yield stress of the compression flange, ksi (MPa)	App. 1.3
F_y	Specified minimum yield stress of the column web, ksi (MPa)	J10.6
F_y	Specified minimum yield stress of HSS member material, ksi (MPa)	K1.1
F_y	Specified minimum yield stress of HSS main member material, ksi (MPa)	K2.1
F_{yb}	Specified minimum yield stress of HSS branch member material, ksi (MPa)	K2.1
F_{ybi}	Specified minimum yield stress of the overlapping branch material, ksi (MPa)	K2.3
F_{ybj}	Specified minimum yield stress of the overlapped branch material, ksi (MPa)	K2.3
F_{yf}	Specified minimum yield stress of the flange, ksi (MPa)	J10.1
F_{ym}	Specified minimum yield stress of the type of steel being used at elevated temperature, ksi (MPa)	App. 4.2
F_{yp}	Specified minimum yield stress of plate, ksi (MPa)	K1.1
F_{yr}	Specified minimum yield stress of reinforcing bars, ksi (MPa)	I2.1
F_{yst}	Specified minimum yield stress of the stiffener material, ksi (MPa)	G3.3
F_{yw}	Specified minimum yield stress of the web, ksi (MPa)	J10.2
G	Shear modulus of elasticity of steel $= 11{,}200$ ksi (77 200 MPa)	E4
ΣH	Story shear produced by the lateral forces used to compute Δ_H, kips (N)	C2.1
H	Overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)	Table D3.1
H	Overall height of rectangular HSS main member, measured in the plane of the connection, in. (mm)	K2.1
H	Flexural constant	E4
H_b	Overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)	K2.1
H_{bi}	Overall depth of the overlapping branch	K2.3
I	Moment of inertia in the place of bending, in.^{4} (mm^4)	C2.1
I	Moment of inertia about the axis of bending, in.^{4} (mm^4)	App. 7.3
I_c	Moment of inertia of the concrete section, in.^{4} (mm^4)	I2.1
I_d	Moment of inertia of the steel deck supported on secondary members, in.^{4} (mm^4)	App. 2.1
I_p	Moment of inertia of primary members, in.^{4} (mm^4)	App. 2.1
I_s	Moment of inertia of secondary members, in.^{4} (mm^4)	App. 2.1

I_s	Moment of inertia of steel shape, in.^{4} (mm^4)	I2.1
I_{sr}	Moment of inertia of reinforcing bars, in.^{4} (mm^4)	I2.1
I_x, I_y	Moment of inertia about the principal axes, in.^{4} (mm^4)	E4
I_y	Out-of-plane moment of inertia, in.^{4} (mm^4)	App. 6.2
I_z	Minor principal axis moment of inertia, in.^{4} (mm^4)	F10.2
I_{yc}	Moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending referred to smaller flange, in.^{4} (mm^4)	F1
J	Torsional constant, in.^{4} (mm^4)	E4
K	Effective length factor determined in accordance with Chapter C	C1.2
K_z	Effective length factor for torsional buckling	E4
K_1	Effective length factor in the plane of bending, calculated based on the assumption of no lateral translation set equal to 1.0 unless analysis indicates that a smaller value may be used	C2.1
K_2	Effective length factor in the plane of bending, calculated based on a sidesway buckling analysis	C2.1
L	Story height, in. (mm)	C2.1
L	Length of the member, in. (mm)	H3
L	Actual length of end-loaded weld, in. (mm)	J2.2
L	Nominal occupancy live load	App. 4.1.4
L	Laterally unbraced length of a member, in. (mm)	E2
L	Span length, in. (mm)	App. 6.2
L	Length of member between work points at truss chord centerlines, in. (mm)	E5
L_b	Length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm)	F2
L_b	Distance between braces, in. (mm)	App. 6.2
L_c	Length of channel shear connector, in. (mm)	I3.2
L_c	Clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)	J3.10
L_e	Total effective weld length of groove and fillet welds to rectangular HSS, in. (mm)	K2.3
L_p	Limiting laterally unbraced length for the limit state of yielding in. (mm)	F2.2
L_p	Column spacing in direction of girder, ft (m)	App. 2
L_{pd}	Limiting laterally unbraced length for plastic analysis, in. (mm)	App. 1.7
L_q	Maximum unbraced length for M_r (the required flexural strength), in. (mm)	App. 6.2
L_r	Limiting laterally unbraced length for the limit state of inelastic lateral-torsional buckling, in. (mm)	F2.2
L_s	Column spacing perpendicular to direction of girder, ft (m)	App. 2.1
L_v	Distance from maximum to zero shear force, in. (mm)	G6

M_A	Absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)	F1
M_a	Required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)	K2.2
M_B	Absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)	F1
M_{br}	Required bracing moment, kip-in. (N-mm)	App. 6.2
M_C	Absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)	F1
$M_{c(x, y)}$	Available flexural strength determined in accordance with Chapter F, kip-in. (N-mm)	H1.1
M_{cx}	Available flexural-torsional strength for strong axis flexure determined in accordance with Chapter F, kip-in. (N-mm)	H1.3
M_e	Elastic lateral-torsional buckling moment, kip-in. (N-mm)	F10.2
M_{lt}	First-order moment under LRFD or ASD load combinations caused by lateral translation of the frame only, kip-in. (N-mm)	C2.1
M_{max}	Absolute value of maximum moment in the unbraced segment, kip-in. (N-mm)	F1
M_n	Nominal flexural strength, kip-in. (N-mm)	F1
M_{nt}	First-order moment using LRFD or ASD load combinations assuming there is no lateral translation of the frame, kip-in. (N-mm)	C2.1
M_p	Plastic bending moment, kip-in. (N-mm)	Table B4.1
M_r	Required second-order flexural strength under LRFD or ASD load combinations, kip-in. (N-mm)	C2.1
M_r	Required flexural strength using LRFD or ASD load combinations, kip-in. (N-mm)	H1
M_r	Required flexural strength in chord, kip-in. (N-mm)	K2.2
$M_{r\text{-}ip}$	Required in-plane flexural strength in branch, kip-in. (N-mm)	K3.2
$M_{r\text{-}op}$	Required out-of-plane flexural strength in branch, kip-in. (N-mm)	K3.2
M_u	Required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)	K2.2
M_y	Yield moment about the axis of bending, kip-in. (N-mm)	Table B4.1
M_1	Smaller moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, kip-in. (N-mm)	C2.1
M_2	Larger moment, calculated from a first-order analysis, at the ends of that portion of the member unbraced in the plane of bending under consideration, kip-in. (N-mm)	C2.1
N	Length of bearing (not less than k for end beam reactions), in. (mm)	J10.2
N	Bearing length of the load, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of the loaded cap plates), in. (mm)	K1.1
N	Number of stress range fluctuations in design life	App. 3.3
N_b	Number of bolts carrying the applied tension	J3.9

N_i	Additional lateral load	C2.2
N_i	Notional lateral load applied at level i, kips (N)	App. 7.3
N_s	Number of slip planes	J3.8
O_v	Overlap connection coefficient	K2.2
P	Pitch, in. per thread (mm per thread)	App. 3.4
P_{br}	Required brace strength, kips (N)	App. 6.2
P_c	Available axial compressive strength, kips (N)	H1.1
P_c	Available tensile strength, kips (N)	H1.2
P_{co}	Available compressive strength out of the plane of bending, kip (N)	H1.3
P_{e1}, P_{e2}	Elastic critical buckling load for braced and unbraced frame, respectively, kips (N)	C2.1
P_{eL}	Euler buckling load, evaluated in the plane of bending, kips (N)	App. 7.3
$P_{l(t,c)}$	First-order axial force using LRFD or ASD load combinations as a result of lateral translation of the frame only (tension or compression), kips (N)	C2.1
$P_{n(t,c)}$	First-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame (tension or compression), kips (N)	C2.1
P_n	Nominal axial strength, kips (N)	D2
P_o	Nominal axial compressive strength without consideration of length effects, kips (N)	I2.1
P_p	Nominal bearing strength of concrete, kips (N)	I2.1
P_r	Required second-order axial strength using LRFD or ASD load combinations, kips (N)	C2.1
P_r	Required axial compressive strength using LRFD or ASD load combinations, kips (N)	C2.2
P_r	Required tensile strength using LRFD or ASD load combinations, kips (N)	H1.2
P_r	Required strength, kips (N)	J10.6
P_r	Required axial strength in branch, kips (N)	K3.2d
P_r	Required axial strength in chord, kips (N)	K2.2
P_u	Required axial strength in compression, kips (N)	App. 1.4
P_y	Member yield strength, kips (N)	C2.2
Q	Full reduction factor for slender compression elements	E7
Q_a	Reduction factor for slender stiffened compression elements	E7.2
Q_f	Chord-stress interaction parameter	K2.2
Q_n	Nominal strength of one stud shear connector, kips (N)	I2.1
Q_s	Reduction factor for slender unstiffened compression elements	E7.1
R	Nominal load due to rainwater or snow, exclusive of the ponding contribution, ksi (MPa)	App. 2.2
R	Seismic response modification coefficient	A1.1
R_a	Required strength (ASD)	B3.4
R_{FIL}	Reduction factor for joints using a pair of transverse fillet welds only	App. 3.3

R_g	Coefficient to account for group effect	I3.2
R_m	Factor in Equation C2-6b dependent on type of system	C2.1
R_m	Cross-section monosymmetry parameter	F1
R_n	Nominal strength, specified in Chapters B through K	B3.3
R_n	Nominal slip resistance, kips (N)	J3.8
R_p	Position effect factor for shear studs	I3.2
R_{pc}	Web plastification factor	F4.1
R_{PJP}	Reduction factor for reinforced or nonreinforced transverse partial-joint-penetration (PJP) groove welds	App. 3.3
R_{pt}	Web plastification factor corresponding to the tension flange yielding limit state	F4.4
R_u	Required strength (LRFD)	B3.3
R_{wl}	Total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.5	J2.4
R_{wt}	Total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.5 without the alternate in Section J2.4 (a)	J2.4
S	Elastic section modulus of round HSS, in.^{3} (mm^3)	F8.2
S	Lowest elastic section modulus relative to the axis of bending, in.^{3} (mm^3)	F12
S	Spacing of secondary members, ft (m)	App. 2.1
S	Nominal snow load	App. 4.1.4
S	Chord elastic section modulus, in.^{3} (mm^3)	K2.2
S_c	Elastic section modulus to the toe in compression relative to the axis of bending, in.^{3} (mm^3).	F10.3
S_{eff}	Effective section modulus about major axis, in.^{3} (mm^3)	F7.2
S_{xt}, S_{xc}	Elastic section modulus referred to tension and compression flanges, respectively, in.^{3} (mm^3)	Table B4.1
S_x, S_y	Elastic section modulus taken about the principal axes, in.^{3} (mm^3)	F2.2, F6
S_y	For channels, taken as the minimum section modulus	F6
T	Nominal forces and deformations due to the design-basis fire defined in Section 4.2.1	App. 4.1.4
T_a	Tension force due to ASD load combinations, kips (kN)	J3.9
T_b	Minimum fastener tension given in Table J3.1 or J3.1M, kips (kN)	J3.8
T_c	Available torsional strength, kip-in. (N-mm)	H3.2
T_n	Nominal torsional strength, kip-in. (N-mm)	H3.1
T_r	Required torsional strength, kip-in. (N-mm)	H3.2
T_u	Tension force due to LRFD load combinations, kips (kN)	J3.9
U	Shear lag factor	D3.3
U	Utilization ratio	K2.2
U_{bs}	Reduction coefficient, used in calculating block shear rupture strength	J4.3
U_p	Stress index	App. 2.2
U_s	Stress index	App. 2.2

V	Required shear force introduced to column, kips (N)	I2.1
V'	Required shear force transferred by shear connectors, kips (N)	I2.1
V_c	Available shear strength, kips (N)	G3.3
V_n	Nominal shear strength, kips (N)	G1
V_r	Required shear strength at the location of the stiffener, kips (N)	G3.3
V_r	Required shear strength using LRFD or ASD load combinations, kips (N)	H3.2
Y_i	Gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i, kips (N)	C2.2
Y_t	Hole reduction coefficient, kips (N)	F13.1
Z	Plastic section modulus about the axis of bending, in.^{3} (mm^3)	F7.1
Z_b	Branch plastic section modulus about the correct axis of bending, in.^{3} (mm^3)	K3.3
$Z_{x,y}$	Plastic section modulus about the principal axes, in.^{3} (mm^3)	F2, F6.1
a	Clear distance between transverse stiffeners, in. (mm)	F13.2
a	Distance between connectors in a built-up member, in. (mm)	E6.1
a	Shortest distance from edge of pin hole to edge of member measured parallel to direction of force, in. (mm)	D5.1
a	Half the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)	App. 3.3
a_w	Ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components	F4.2
b	Outside width of leg in compression, in. (mm)	F10.3
b	Full width of longest angle leg, in. (mm)	E7.1
b	Width of unstiffened compression element; for flanges of I-shaped members and tees, the width b is half the full-flange width, b_f; for legs of angles and flanges of channels and zees, the width b is the full nominal dimension; for plates, the width b is the distance from the free edge to the first row of fasteners or line of welds, or the distance between adjacent lines of fasteners or lines of welds; for rectangular HSS, the width b is the clear distance between the webs less the inside corner radius on each side, in. (mm)	B4.1, B4.2
b	Width of the angle leg resisting the shear force, in. (mm)	G4
b_{cf}	Width of column flange, in. (mm)	J10.6
b_e	Reduced effective width, in. (mm)	E7.2
b_{eff}	Effective edge distance; the distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force, in. (mm)	D5.1
b_{eoi}	Effective width of the branch face welded to the chord	K2.3
b_{eov}	Effective width of the branch face welded to the overlapped brace	K2.3
b_f	Flange width, in. (mm)	B4.1
b_{fc}	Compression flange width, in. (mm)	F4.2
b_{ft}	Width of tension flange, in. (mm)	G3.1

b_l	Longer leg of angle, in. (mm)	E5
b_s	Shorter leg of angle, in. (mm)	E5
b_s	Stiffener width for one-sided stiffeners, in. (mm)	App. 6.2
d	Nominal fastener diameter, in. (mm)	J3.3
d	Full nominal depth of the section, in. (mm)	B4.1
d	Full nominal depth of tee, in. (mm)	E7.1
d	Depth of rectangular bar, in. (mm)	F11.2
d	Full nominal depth of section, in. (mm)	B4.1
d	Full nominal depth of tee, in. (mm)	E7.1
d	Diameter, in. (mm)	J7
d	Pin diameter, in. (mm)	D5.1
d	Roller diameter, in. (mm)	J7
d_b	Beam depth, in. (mm)	J10.6
d_b	Nominal diameter (body or shank diameter), in. (mm)	App. 3.4
d_c	Column depth, in. (mm)	J10.6
e	Eccentricity in a truss connection, positive being away from the branches, in. (mm)	K2.1
$e_{mid\text{-}ht}$	Distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the load bearing direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), in. (mm)	I3.2
f_a	Required axial stress at the point of consideration using LRFD or ASD load combinations, ksi (MPa)	H2
$f_{b(w,z)}$	Required flexural stress at the point of consideration (major axis, minor axis) using LRFD or ASD load combinations, ksi (MPa)	H2
f'_c	Specified minimum compressive strength of concrete, ksi (MPa)	I1.1
f'_{cm}	Specified minimum compressive strength of concrete at elevated temperatures, ksi (MPa)	App. 4.2
f_o	Stress due to $D + R$ (the nominal dead load + the nominal load due to rainwater or snow exclusive of the ponding contribution), ksi (MPa)	App. 2.2
f_v	Required shear strength per unit area, ksi (MPa)	J3.7
g	Transverse center-to-center spacing (gage) between fastener gage lines, in. (mm)	B3.13
g	Gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)	K2.1
h	Clear distance between flanges less the fillet or corner radius for rolled shapes; for built-up sections, the distance between adjacent lines of fasteners or the clear distance between flanges when welds are used; for tees, the overall depth; for rectangular HSS, the clear distance between the flanges less the inside corner radius on each side, in. (mm)	B4.2
h	Distance between centroids of individual components perpendicular to the member axis of buckling, in. (mm)	E6.1

h_c	Twice the distance from the centroid to the following: the inside face of the compression flange less the fillet or corner radius, for rolled shapes; the nearest line of fasteners at the compression flange or the inside faces of the compression flange when welds are used, for built-up sections, in. (mm)	B4.2
h_o	Distance between flange centroids, in. (mm)	F2.2
h_p	Twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used, in. (mm)	B4.2
h_{sc}	Hole factor	J3.8
j	Factor defined by Equation G2-6 for minimum moment of inertia for a transverse stiffener	G2.2
k	Distance from outer face of flange to the web toe of fillet, in. (mm)	J10.2
k	Outside corner radius of the HSS, which is permitted to be taken as $1.5t$ if unknown, in. (mm)	K1.3
k_c	Coefficient for slender unstiffened elements, in. (mm)	Table B4.1
k_s	Slip-critical combined tension and shear coefficient	J3.9
k_v	Web plate buckling coefficient	G2.1
l	Largest laterally unbraced length along either flange at the point of load, in. (mm)	J10.4
l	Length of bearing, in. (mm)	J7
l	Length of connection in the direction of loading, in. (mm)	Table D3.1
n	Number of nodal braced points within the span	App. 6.2
n	Threads per inch (per mm)	App. 3.4
p	Ratio of element i deformation to its deformation at maximum stress	J2.4
p	Projected length of the overlapping branch on the chord	K2.2
q	Overlap length measured along the connecting face of the chord beneath the two branches	K2.2
r	Governing radius of gyration, in. (mm)	E2
r_{crit}	Distance from instantaneous center of rotation to weld element with minimum Δ_u/r_i ratio, in. (mm)	J2.4
r_i	Minimum radius of gyration of individual component in a built-up member, in. (mm)	E6.1
r_{ib}	Radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, in. (mm)	E6.1
$\overline{r_o}$	Polar radius of gyration about the shear center, in. (mm)	E4
r_t	Radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone	F4.2
r_{ts}	Effective radius of gyration used in the determination of L_r for the lateral-torsional buckling limit state for major axis bending of doubly symmetric compact I-shaped members and channels	F2.2
r_x	Radius of gyration about geometric axis parallel to connected leg, in. (mm)	E5

r_y	Radius of gyration about y-axis, in. (mm)	E4
r_z	Radius of gyration for the minor principal axis, in. (mm)	E5
s	Longitudinal center-to-center spacing (pitch) of any two consecutive holes, in. (mm)	B3.13
t	Thickness of element, in. (mm)	B4.2
t	Wall thickness, in. (mm)	E7.2
t	Angle leg thickness, in. (mm)	F10.2
t	Width of rectangular bar parallel to axis of bending, in. (mm)	F11.2
t	Thickness of connected material, in. (mm)	J3.10
t	Thickness of plate, in. (mm)	D5.1
t	Design wall thickness for HSS equal to 0.93 times the nominal wall thickness for ERW HSS and equal to the nominal wall thickness for SAW HSS, in. (mm)	B3.12
t	Total thickness of fillers, in. (mm)	J5
t	Design wall thickness of HSS main member, in. (mm)	K2.1
t_b	Design wall thickness of HSS branch member, in. (mm)	K2.1
t_{bi}	Thickness of the overlapping branch, in. (mm)	K2.3
t_{bj}	Thickness of the overlapped branch, in. (mm)	K2.3
t_{cf}	Thickness of the column flange, in. (mm)	J10.6
t_f	Thickness of the loaded flange, in. (mm)	J10.1
t_f	Flange thickness of channel shear connector, in. (mm)	I3.2
t_{fc}	Compression flange thickness, in. (mm)	F4.2
t_p	Thickness of plate, in. (mm)	K1.1
t_p	Thickness of tension loaded plate, in. (mm)	App. 3.3
t_p	Thickness of the attached transverse plate, in. (mm)	K2.3
t_s	Web stiffener thickness, in. (mm)	App. 6.2
t_w	Web thickness of channel shear connector, in. (mm)	I3.2
t_w	Beam web thickness, in. (mm)	App. 6.3
t_w	Web thickness, in. (mm)	Table B4.1
t_w	Column web thickness, in. (mm)	J10.6
t_w	Thickness of element, in. (mm)	E7.1
w	Width of cover plate, in. (mm)	F13.3
w	Weld leg size, in. (mm)	J2.2
w	Subscript relating symbol to major principal axis bending	
w	Plate width, in. (mm)	Table D3.1
w	Leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)	App. 3.3
w_c	Weight of concrete per unit volume ($90 \leq w_c \leq 155$ lbs/ft^3 or $1500 \leq w_c \leq 2500$ kg/m^3)	I2.1
w_r	Average width of concrete rib or haunch, in. (mm)	I3.2
x	Subscript relating symbol to strong axis	
x_o, y_o	Coordinates of the shear center with respect to the centroid, in. (mm)	E4
$\overline{x}$	Connection eccentricity, in. (mm)	Table D3.1

y	Subscript relating symbol to weak axis	
z	Subscript relating symbol to minor principal axis bending	
α	Factor used in B2 equation	C2.1
α	Separation ratio for built-up compression members $= \dfrac{h}{2r_{ib}}$	E6.1
β	Reduction factor given by Equation J2-1	J2.2
β	Width ratio; the ratio of branch diameter to chord diameter for round HSS; the ratio of overall branch width to chord width for rectangular HSS	K2.1
β_T	Brace stiffness requirement excluding web distortion, kip-in./radian (N-mm/radian)	App. 6.2
β_{br}	Required brace stiffness	App. 6.2
β_{eff}	Effective width ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width	K2.1
β_{eop}	Effective outside punching parameter	K2.3
β_{sec}	Web distortional stiffness, including the effect of web transverse stiffeners, if any, kip-in./radian (N-mm/radian)	App. 6.2
β_w	Section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression	F10.2
Δ	First-order interstory drift due to the design loads, in. (mm)	C2.2
Δ_H	First-order interstory drift due to lateral forces, in. (mm)	C2.1
Δ_i	Deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in. (mm)	J2.4
Δ_m	Deformation of weld element at maximum stress, in. (mm)	J2.4
Δ_u	Deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, in. (mm)	J2.4
γ	Chord slenderness ratio; the ratio of one-half the diameter to the wall thickness for round HSS; the ratio of one-half the width to wall thickness for rectangular HSS	K2.1
ζ	Gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord for rectangular HSS	K2.1
η	Load length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width	K2.1
λ	Slenderness parameter	F3
λ_p	Limiting slenderness parameter for compact element	B4
λ_{pf}	Limiting slenderness parameter for compact flange	F3
λ_{pw}	Limiting slenderness parameter for compact web	F4
λ_r	Limiting slenderness parameter for noncompact element	B4
λ_{rf}	Limiting slenderness parameter for noncompact flange	F3
λ_{rw}	Limiting slenderness parameter for noncompact web	F4
μ	Mean slip coefficient for class A or B surfaces, as applicable, or as established by tests	J3.8

ϕ	Resistance factor, specified in Chapters B through K	B3.3
ϕ_B	Resistance factor for bearing on concrete	I2.1
ϕ_b	Resistance factor for flexure	F1
ϕ_c	Resistance factor for compression	E1
ϕ_c	Resistance factor for axially loaded composite columns	I2.1b
ϕ_{sf}	Resistance factor for shear on the failure path	D5.1
ϕ_T	Resistance factor for torsion	H3.1
ϕ_t	Resistance factor for tension	D2
ϕ_v	Resistance factor for shear	G1
Ω	Safety factor	B3.4
Ω_B	Safety factor for bearing on concrete	I2.1
Ω_b	Safety factor for flexure	F1
Ω_c	Safety factor for compression	E1
Ω_c	Safety factor for axially loaded composite columns	I2.1b
Ω_{sf}	Safety factor for shear on the failure path	D5.1
Ω_t	Safety factor for torsion	H3.1
Ω_t	Safety factor for tension	D2
Ω_v	Safety factor for shear	G1
ρ_{sr}	Minimum reinforcement ratio for longitudinal reinforcing	I2.1
θ	Angle of loading measured from the weld longitudinal axis, degrees	J2.4
θ	Acute angle between the branch and chord, degrees	K2.1
ε_{cu}	Strain corresponding to compressive strength, f'_c	App. 4.2
τ_b	Parameter for reduced flexural stiffness using the direct analysis method	App. 7.3

GLOSSARY

Terms that appear in this Glossary are *italicized* throughout the Specification, where they first appear within a sub-section.

Notes:

(1) Terms designated with † are common AISI-AISC terms that are coordinated between the two standards developers.
(2) Terms designated with * are usually qualified by the type of *load effect,* for example, *nominal tensile strength*, *available compressive strength*, *design flexural strength.*
(3) Terms designated with ** are usually qualified by the type of component, for example, web *local buckling*, flange *local bending.*

Allowable strength * †. *Nominal strength* divided by the *safety factor*, R_n / Ω.

Allowable stress. Allowable strength divided by the appropriate section property, such as section modulus or cross-section area.

Amplification factor. Multiplier of the results of *first-order analysis* to reflect *second-order* effects.

Applicable building code†. Building code under which the structure is designed.

ASD (Allowable Strength Design)†. Method of proportioning *structural components* such that the *allowable strength* equals or exceeds the *required strength* of the component under the action of the *ASD load combinations.*

ASD load combination†. Load combination in the *applicable building code* intended for allowable strength design (*allowable stress design*).

Authority having jurisdiction. Organization, political subdivision, office or individual charged with the responsibility of administering and enforcing the provisions of the *applicable building code.*

*Available strength**†. *Design strength* or *allowable strength,* as appropriate.

*Available stress**. *Design stress* or *allowable stress*, as appropriate.

Average rib width. Average width of the rib of a corrugation in a *formed steel deck.*

Batten plate. Plate rigidly connected to two parallel components of a built-up *column* or *beam* designed to transmit shear between the components.

Beam. Structural member that has the primary function of resisting bending moments.

Beam-column. Structural member that resists both axial force and bending moment.

Bearing. In a bolted connection, *limit state* of shear forces transmitted by the bolt to the connection elements.

Bearing (local compressive yielding). Limit state of *local* compressive *yielding* due to the action of a member bearing against another member or surface.

Bearing-type connection. Bolted *connection* where shear forces are transmitted by the bolt bearing against the connection elements.

Block shear rupture. In a connection, *limit state* of tension fracture along one path and shear yielding or shear fracture along another path.

Braced frame†. An essentially vertical truss system that provides resistance to lateral forces and provides stability for the *structural system*.

Branch face. Wall of *HSS branch member.*

Branch member. For *HSS connections*, member that terminates at a *chord member* or *main member*.

Buckling. Limit state of sudden change in the geometry of a structure or any of its elements under a critical loading condition.

Buckling strength. Nominal strength for *buckling* or *instability limit states*.

Built-up member, cross-section, section, shape. Member, cross-section, section or shape fabricated from structural steel elements that are welded or bolted together.

Camber. Curvature fabricated into a beam or truss so as to compensate for deflection induced by loads.

Charpy V-Notch impact test. Standard dynamic test measuring notch toughness of a specimen.

Chord member. For *HSS*, primary member that extends through a truss *connection*.

Cladding. Exterior covering of structure.

Cold-formed steel structural member†. Shape manufactured by press-braking blanks sheared from sheets, cut lengths of coils or plates, or by roll forming cold- or hot-rolled coils or sheets; both forming operations being performed at ambient room temperature, that is, without manifest addition of heat such as would be required for hot forming.

Column. Structural member that has the primary function of resisting axial force.

Combined system. Structure comprised of two or more lateral load-resisting systems of different type.

Compact section. Section capable of developing a fully plastic stress distribution and possessing a *rotation capacity* of approximately three before the onset of local buckling.

Complete-joint-penetration groove weld (CJP). Groove weld in which weld metal extends through the joint thickness, except as permitted for *HSS connections*.

Composite. Condition in which steel and concrete elements and members work as a unit in the distribution of internal forces.

Concrete crushing. Limit state of compressive failure in concrete having reached the ultimate strain.

Concrete haunch. Section of solid concrete that results from stopping the deck on each side of the girder in a *composite* floor system constructed using a *formed steel deck.*

Concrete-encased beam. Beam totally encased in concrete cast integrally with the slab.

Connection†. Combination of structural elements and *joints* used to transmit forces between two or more members.

Cope. Cutout made in a structural member to remove a flange and conform to the shape of an intersecting member.

Cover plate. Plate welded or bolted to the flange of a member to increase cross-sectional area, section modulus or moment of inertia.

Cross connection. HSS connection in which forces in *branch members* or connecting elements transverse to the *main member* are primarily equilibrated by forces in other branch members or connecting elements on the opposite side of the main member.

*Design load**†. Applied *load* determined in accordance with either *LRFD load combinations* or *ASD load combinations*, whichever is applicable.

*Design strength**†. *Resistance factor* multiplied by the *nominal strength*, ϕR_n.

Design stress range. Magnitude of change in stress due to the repeated application and removal of service live *loads*. For locations subject to stress reversal it is the algebraic difference of the peak stresses.

*Design stress**. *Design strength* divided by the appropriate section property, such as section modulus or cross section area.

Design wall thickness. *HSS* wall thickness assumed in the determination of section properties.

Diagonal bracing. Inclined structural member carrying primarily axial force in a *braced frame.*

Diagonal stiffener. Web stiffener at column panel zone oriented diagonally to the flanges, on one or both sides of the web.

Diaphragm plate. Plate possessing in-plane shear stiffness and strength, used to transfer forces to the supporting elements.

Diaphragm†. Roof, floor or other membrane or bracing system that transfers in-plane forces to the lateral force resisting system.

Direct analysis method. Design method for stability that captures the effects of residual stresses and initial out-of-plumbness of frames by reducing stiffness and applying *notional loads* in a *second-order analysis.*

Direct bond interaction. Mechanism by which force is transferred between steel and concrete in a *composite* section by bond stress.

Distortional failure. Limit state of an *HSS* truss connection based on distortion of a rectangular *HSS* chord member into a rhomboidal shape.

Distortional stiffness. Out-of-plane flexural stiffness of web.

Double curvature. Deformed shape of a beam with one or more inflection points within the span.

Double-concentrated forces. Two equal and opposite forces that form a couple on the same side of the loaded member.

Doubler. Plate added to, and parallel with, a *beam* or *column* web to increase resistance to concentrated forces.

Drift. Lateral deflection of structure.

Effective length factor, K. Ratio between the *effective length* and the unbraced length of the member.

Effective length. Length of an otherwise identical *column* with the same strength when analyzed with pinned end conditions.

Effective net area. Net area modified to account for the effect of shear lag.

Effective section modulus. Section modulus reduced to account for buckling of slender compression elements.

Effective width. Reduced width of a plate or slab with an assumed uniform stress distribution which produces the same effect on the behavior of a structural member as the actual plate or slab width with its nonuniform stress distribution.

Elastic analysis. Structural analysis based on the assumption that the structure returns to its original geometry on removal of the *load*.

Encased composite column. Composite column consisting of a *structural concrete column* and one or more embedded steel shapes.

End panel. Web panel with an adjacent panel on one side only.

End return. Length of *fillet weld* that continues around a corner in the same plane.

Engineer of record. Licensed professional responsible for sealing the contract documents.

Expansion rocker. Support with curved surface on which a member bears that can tilt to accommodate expansion.

Expansion roller. Round steel bar on which a member bears that can roll to accommodate expansion.

Eyebar. Pin-connected tension member of uniform thickness, with forged or thermally cut head of greater width than the body, proportioned to provide approximately equal strength in the head and body.

Factored load†. Product of a *load factor* and the *nominal load.*

Fastener. Generic term for bolts, rivets, or other connecting devices.

Fatigue. Limit state of crack initiation and growth resulting from repeated application of live *loads*.

Faying surface. Contact surface of connection elements transmitting a shear force.

Filled composite column. Composite column consisting of a shell of *HSS* or *pipe* filled with *structural concrete*.

Filler metal. Metal or alloy to be added in making a welded joint.

Filler. Plate used to build up the thickness of one component.

Fillet weld reinforcement. Fillet welds added to *groove welds*.

Fillet weld. Weld of generally triangular cross section made between intersecting surfaces of elements.

First-order analysis. Structural analysis in which equilibrium conditions are formulated on the undeformed structure; *second-order effects* are neglected.

Fitted bearing stiffener. Stiffener used at a support or concentrated *load* that fits tightly against one or both flanges of a *beam* so as to transmit load through bearing.

Flare bevel groove weld. Weld in a groove formed by a member with a curved surface in contact with a planar member.

Flare V-groove weld. Weld in a groove formed by two members with curved surfaces.

Flat width. Nominal width of rectangular *HSS* minus twice the outside corner radius. In absence of knowledge of the corner radius, the flat width may be taken as the total section width minus three times the thickness.

Flexural buckling. Buckling mode in which a compression member deflects laterally without twist or change in cross-sectional shape.

Flexural-torsional buckling†. Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Force. Resultant of distribution of stress over a prescribed area.

Formed section. See *cold-formed steel structural member.*

Formed steel deck. In composite construction, *steel* cold formed into a decking profile used as a permanent concrete form.

Fully restrained moment connection. Connection capable of transferring moment with negligible rotation between connected members.

Gage. Transverse center-to-center spacing of *fasteners*.

Gap connection. HSS truss *connection* with a gap or space on the *chord* face between intersecting *branch members*.

General collapse. Limit state of *chord plastification* of opposing sides of a round *HSS chord member* at a cross-connection.

Geometric axis. Axis parallel to web, flange or angle leg.

Girder filler. Narrow piece of *sheet steel* used as a fill between the edge of a deck sheet and the flange of a girder in a *composite* floor system constructed using a *formed steel deck*.

Girder. See *Beam*.

Girt†. Horizontal structural member that supports wall panels and is primarily subjected to bending under horizontal *loads*, such as wind load.

Gouge. Relatively smooth surface groove or cavity resulting from plastic deformation or removal of material.

Gravity axis. Axis through the center of gravity of a member along its length.

Gravity frame. Portion of the framing system not included in the *lateral load resisting system*.

Gravity load. Load, such as that produced by dead and live loads, acting in the downward direction.

Grip (of bolt). Thickness of material through which a bolt passes.

Groove weld. Weld in a groove between connection elements. See also AWS D1.1.

Gusset plate. Plate element connecting truss members or a strut or brace to a *beam* or *column*.

Horizontal shear. Force at the interface between steel and concrete surfaces in a *composite beam*.

HSS. Square, rectangular or round hollow structural steel section produced in accordance with a *pipe* or tubing product specification.

User Note: A pipe can be designed using the same design rules for round *HSS* sections as long as it conforms to ASTM A53 Class B and the appropriate parameters are used in the design.

Inelastic analysis. *Structural analysis* that takes into account inelastic material behavior, including *plastic analysis*.

In-plane instability. Limit state of a *beam-column* bent about its major axis while *lateral buckling* or *lateral-torsional buckling* is prevented by *lateral bracing*.

Instability. Limit state reached in the loading of a structural component, frame or structure in which a slight disturbance in the *loads* or geometry produces large displacements.

Joint eccentricity. For *HSS* truss *connection*, perpendicular distance from *chord member* center of gravity to intersection of *branch member* work points.

Joint†. Area where two or more ends, surfaces, or edges are attached. Categorized by type of *fastener* or weld used and method of force transfer.

K-connection. HSS connection in which forces in *branch members* or connecting elements transverse to the *main member* are primarily equilibriated by forces in other branch members or connecting elements on the same side of the main member.

Lacing. Plate, angle or other steel shape, in a lattice configuration, that connects two steel shapes together.

Lap joint. Joint between two overlapping connection elements in parallel planes.

Lateral bracing. Diagonal bracing, *shear walls* or equivalent means for providing in-plane lateral stability.

Lateral load resisting system. Structural system designed to resist lateral loads and provide stability for the structure as a whole.

Lateral load. Load, such as that produced by wind or earthquake effects, acting in a lateral direction.

Lateral-torsional buckling. Buckling mode of a flexural member involving deflection normal to the plane of bending occurring simultaneously with twist about the shear center of the cross-section.

Leaning column. Column designed to carry *gravity loads* only, with *connections* that are not intended to provide resistance to *lateral loads.*

Length effects. Consideration of the reduction in strength of a member based on its *unbraced length.*

Limit state. Condition in which a structure or component becomes unfit for service and is judged either to be no longer useful for its intended function (*serviceability limit state*) or to have reached its ultimate load-carrying capacity (*strength limit state*).

Load†. Force or other action that results from the weight of building materials, occupants and their possessions, environmental effects, differential movement, or restrained dimensional changes.

Load effect†. Forces, stresses and deformations produced in a *structural component* by the applied *loads*.

Load factor†. Factor that accounts for deviations of the *nominal load* from the actual *load*, for uncertainties in the analysis that transforms the load into a *load effect* and for the probability that more than one extreme load will occur simultaneously.

*Local bending**. Limit state* of large deformation of a flange under a concentrated tensile force.

*Local buckling**. Limit state* of buckling of a compression element within a cross section.

*Local crippling**. Limit state* of local failure of web plate in the immediate vicinity of a concentrated *load* or reaction.

*Local yielding**. Yielding* that occurs in a local area of an element.

LRFD (Load and Resistance Factor Design)†. Method of proportioning *structural components* such that the *design strength* equals or exceeds the *required strength* of the component under the action of the *LRFD load combinations*.

LRFD load combination†. Load combination in the *applicable building code* intended for strength design (*load and resistance factor design).*

Main member. For *HSS* connections, *chord member*, column or other HSS member to which *branch members* or other connecting elements are attached.

Mechanism. Structural system that includes a sufficient number of real hinges, plastic hinges or both, so as to be able to articulate in one or more rigid body modes.

Mill scale. Oxide surface coating on steel formed by the hot rolling process.

Milled surface. Surface that has been machined flat by a mechanically guided tool to a flat, smooth condition.

Moment connection. Connection that transmits bending moment between connected members.

Moment frame†. Framing system that provides resistance to lateral loads and provides stability to the *structural system*, primarily by shear and flexure of the framing members and their connections.

Net area. Gross area reduced to account for removed material.

Nodal brace. Brace that prevents lateral movement or twist independently of other braces at adjacent brace points (see *relative brace*).

Nominal dimension. Designated or theoretical dimension, as in the tables of section properties.

Nominal load†. Magnitude of the *load* specified by the *applicable building code*.

Nominal rib height. Height of *formed steel deck* measured from the underside of the lowest point to the top of the highest point.

*Nominal strength**†. Strength of a structure or component (without the *resistance factor* or *safety factor* applied) to resist *load effects*, as determined in accordance with this *Specification.*

Noncompact section. Section that can develop the *yield stress* in its compression elements before *local buckling* occurs, but cannot develop a *rotation capacity* of three.

Nondestructive testing. Inspection procedure wherein no material is destroyed and integrity of the material or component is not affected.

Notch toughness. Energy absorbed at a specified temperature as measured in the Charpy V-Notch test.

Notional load. Virtual *load* applied in a *structural analysis* to account for destabilizing effects that are not otherwise accounted for in the design provisions.

Out-of-plane buckling. Limit state of a beam-column bent about its major axis while lateral buckling or *lateral-torsional buckling* is not prevented by lateral bracing.

Overlap connection. HSS truss *connection* in which intersecting *branch members* overlap.

Panel zone. Web area of beam-to-column connection delineated by the extension of beam and column flanges through the connection, transmitting moment through a shear panel.

Partial-joint-penetration groove weld (PJP). Groove weld in which the penetration is intentionally less than the complete thickness of the connected element.

Partially restrained moment connection. Connection capable of transferring moment with rotation between connected members that is not negligible.

Percent elongation. Measure of ductility, determined in a tensile test as the maximum elongation of the gage length divided by the original gage length.

Permanent load†. *Load* in which variations over time are rare or of small magnitude. All other *loads* are *variable loads*.

Pipe. See *HSS*.

Pitch. Longitudinal center-to-center spacing of fasteners. Center-to-center spacing of bolt threads along axis of bolt.

Plastic analysis. Structural analysis based on the assumption of rigid-plastic behavior, in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress.

Plastic hinge. Yielded zone that forms in a structural member when the *plastic moment* is attained. The member is assumed to rotate further as if hinged, except that such rotation is restrained by the *plastic moment*.

Plastic moment. Theoretical resisting moment developed within a fully yielded cross section.

Plastic stress distribution method. Method for determining the stresses in a composite member assuming that the steel section and the concrete in the cross section are fully plastic.

Plastification. In an HSS connection, *limit state* based on an out-of-plane flexural yield line *mechanism* in the *chord* at a *branch member* connection.

Plate girder. Built-up *beam*.

Plug weld. Weld made in a circular hole in one element of a joint fusing that element to another element.

Ponding. Retention of water due solely to the deflection of flat roof framing.

Post-buckling strength. Load or force that can be carried by an element, member, or frame after initial buckling has occurred.

Pretensioned joint. Joint with high-strength bolts tightened to the specified minimum pretension.

Properly developed. Reinforcing bars detailed to yield in a ductile manner before crushing of the concrete occurs. Bars meeting the provisions of ACI 318 insofar as development length, spacing and cover shall be deemed to be properly developed.

Prying action. Amplification of the tension force in a bolt caused by leverage between the point of applied load, the bolt and the reaction of the connected elements.

Punching load. Component of *branch member* force perpendicular to a *chord*.

Purlin†. Horizontal structural member that supports roof deck and is primarily subjected to bending under vertical loads such as snow, wind or dead loads.

P-δ effect. Effect of *loads* acting on the deflected shape of a member between joints or nodes.

P-Δ effect. Effect of *loads* acting on the displaced location of joints or nodes in a structure. In tiered building structures, this is the effect of *loads* acting on the laterally displaced location of floors and roofs.

Quality assurance. System of shop and field activities and controls implemented by the owner or his/her designated representative to provide confidence to the owner and the building authority that quality requirements are implemented.

Quality control. System of shop and field controls implemented by the fabricator and erector to ensure that contract and company fabrication and erection requirements are met.

Rational engineering analysis†. *Analysis* based on theory that is appropriate for the situation, relevant test data if available, and sound engineering judgment.

Reentrant. In a *cope* or weld access hole, a cut at an abrupt change in direction in which the exposed surface is concave.

Relative brace. Brace that controls the relative movement of two adjacent brace points along the length of a *beam* or *column* or the relative lateral displacement of two stories in a frame (see *nodal brace*).

*Required strength**†. Forces, stresses and deformations acting on the *structural component*, determined by either *structural analysis*, for the *LRFD* or *ASD load combinations*, as appropriate, or as specified by this *Specification* or Standard.

Resistance factor, ϕ†. Factor that accounts for unavoidable deviations of the *nominal strength* from the actual strength and for the manner and consequences of failure.

Reverse curvature. See *double curvature.*

Root of joint. Portion of a *joint* to be welded where the members are closest to each other.

Rotation capacity. Incremental angular rotation that a given shape can accept prior to excessive load shedding, defined as the ratio of the inelastic rotation attained to the idealized elastic rotation at first yield.

Rupture strength. In a *connection*, strength limited by tension or shear rupture.

Safety factor, Ω†. Factor that accounts for deviations of the actual strength from the nominal strength, deviations of the actual *load* from the *nominal load*, uncertainties in the analysis that transforms the load into a *load effect*, and for the manner and consequences of failure.

Second-order analysis. Structural analysis in which equilibrium conditions are formulated on the deformed structure; *second-order effects* (both P-δ and P-Δ, unless specified otherwise) are included.

Second-order effect. Effect of *loads* acting on the deformed configuration of a structure; includes P-δ *effect* and P-Δ *effect.*

Seismic response modification coefficient. Factor that reduces seismic *load effects* to strength level.

Service load combination. Load combination under which serviceability limit states are evaluated.

Service load†. *Load* under which *serviceability limit states* are evaluated.

Serviceability limit state. Limiting condition affecting the ability of a structure to preserve its appearance, maintainability, durability or the comfort of its occupants or function of machinery, under normal usage.

Shear buckling. Buckling mode in which a plate element, such as the web of a beam, deforms under pure shear applied in the plane of the plate.

Shear connector. Headed stud, channel, plate or other shape welded to a steel member and embedded in concrete of a *composite member* to transmit shear forces at the interface between the two materials.

Shear connector strength. Limit state of reaching the strength of a *shear connector,* as governed by the connector bearing against the concrete in the slab or by the *tensile strength* of the connector.

Shear rupture. Limit state *of rupture (fracture) due to shear.*

Shear wall†. Wall that provides resistance to lateral loads in the plane of the wall and provides stability for the structural system.

Shear yielding. Yielding that occurs due to shear.

Shear yielding (punching). In an HSS connection, *limit state* based on out-of-plane shear strength of the *chord* wall to which *branch members* are attached.

Sheet steel. In a composite floor system, steel used for closure plates or miscellaneous trimming in a *formed steel deck*.

Shim. Thin layer of material used to fill a space between faying or bearing surfaces.

Sidesway buckling. *Limit state* of lateral buckling of the tension flange opposite the location of a concentrated compression force.

Sidewall crippling. Limit state of *web crippling* of the sidewalls of a *chord member* at a *HSS* truss *connection*.

Sidewall crushing. Limit state based on bearing strength of *chord member* sidewall in *HSS* truss *connection*.

Simple connection. Connection that transmits negligible bending moment between connected members.

Single-concentrated force. Tensile or compressive force applied normal to the flange of a member.

Single curvature. Deformed shape of a beam with no inflection point within the span.

Slender-element section. Cross section possessing plate components of sufficient slenderness such that *local buckling* in the elastic range will occur.

Slip. In a bolted connection, *limit state* of relative motion of connected parts prior to the attainment of the *available strength* of the connection.

Slip-critical connection. Bolted *connection* designed to resist movement by friction on the faying surface of the connection under the clamping forces of the bolts.

Slot weld. Weld made in an elongated hole fusing an element to another element.

Snug-tightened joint. Joint with the connected plies in firm contact as specified in Chapter J.

Specified minimum tensile strength. Lower limit of *tensile strength* specified for a material as defined by ASTM.

Specified minimum yield stress†. Lower limit of *yield stress* specified for a material as defined by ASTM.

Splice. Connection between two structural elements joined at their ends to form a single, longer element.

Stability. Condition reached in the loading of a structural component, frame or structure in which a slight disturbance in the *loads* or geometry does not produce large displacements.

Stiffened element. Flat compression element with adjoining out-of-plane elements along both edges parallel to the direction of loading.

Stiffener. Structural element, usually an angle or plate, attached to a *member* to distribute *load*, transfer shear or prevent buckling.

Stiffness. Resistance to deformation of a member or structure, measured by the ratio of the applied force (or moment) to the corresponding displacement (or rotation).

Strain compatibility method. Method for determining the stresses in a composite member considering the stress-strain relationships of each material and its location with respect to the neutral axis of the cross section.

Strength limit state. Limiting condition affecting the safety of the structure, in which the ultimate load-carrying capacity is reached.

Stress. Force per unit area caused by axial force, moment, shear or torsion.

Stress concentration. Localized stress considerably higher than average (even in uniformly loaded cross sections of uniform thickness) due to abrupt changes in geometry or localized loading.

Strong axis. Major principal centroidal axis of a cross section.

Structural analysis†. Determination of *load effects* on members and *connections* based on principles of structural mechanics.

Structural component†. Member, connector, connecting element or assemblage.

Structural steel. Steel elements as defined in Section 2.1 of the AISC *Code of Standard Practice for Steel Buildings and Bridges*.

Structural system. An assemblage of load-carrying components that are joined together to provide interaction or interdependence.

T-connection. HSS connection in which the *branch member* or connecting element is perpendicular to the *main member* and in which forces transverse to the main member are primarily equilibriated by shear in the main member.

Tensile rupture. Limit state of rupture (fracture) due to tension.

Tensile strength (of material)†. Maximum tensile stress that a material is capable of sustaining as defined by ASTM.

Tensile strength (of member). Maximum tension force that a member is capable of sustaining.

Tensile yielding. Yielding *that occurs due to tension.*

Tension and shear rupture. In a bolt, *limit state* of rupture (fracture) due to simultaneous tension and shear *force*.

Tension field action. Behavior of a panel under shear in which diagonal tensile forces develop in the web and compressive forces develop in the *transverse stiffeners* in a manner similar to a Pratt truss.

Thermally cut. Cut with gas, plasma or laser.

Tie plate. Plate element used to join two parallel components of a *built-up column*, girder or strut rigidly connected to the parallel components and designed to transmit shear between them.

Toe of fillet. Junction of a fillet weld face and base metal. Tangent point of a rolled section fillet.

Torsional bracing. Bracing resisting twist of a *beam* or *column*.

Torsional buckling. Buckling mode in which a compression member twists about its shear center axis.

Torsional yielding. Yielding that occurs due to torsion.

Transverse reinforcement. Steel reinforcement in the form of closed ties or welded wire fabric providing confinement for the concrete surrounding the steel shape core in an *encased concrete composite column*.

Transverse stiffener. Web *stiffener* oriented perpendicular to the flanges, attached to the web.

Tubing. See *HSS*.

Turn-of-nut method. Procedure whereby the specified pretension in high-strength bolts is controlled by rotating the fastener component a predetermined amount after the bolt has been snug tightened.

Unbraced length. Distance between braced points of a member, measured between the centers of gravity of the bracing members.

Uneven load distribution. In an *HSS connection*, condition in which the load is not distributed through the cross section of connected elements in a manner that can be readily determined.

Unframed end. The end of a member not restrained against rotation by stiffeners or connection elements.

Unstiffened element. Flat compression element with an adjoining out-of-plane element along one edge parallel to the direction of loading.

Variable load†. *Load* not classified as *permanent load*.

Vertical bracing system. System of *shear walls*, *braced frames* or both, extending through one or more floors of a building.

Weak axis. Minor principal centroidal axis of a cross section.

Weathering steel. High-strength, low-alloy steel that, with suitable precautions, can be used in normal atmospheric exposures (not marine) without protective paint coating.

Web buckling. Limit state of lateral instability of a web.

Web compression buckling. Limit state of out-of-plane compression buckling of the web due to a concentrated compression force.

Web sidesway buckling. Limit state of lateral buckling of the tension flange opposite the location of a concentrated compression force.

Weld metal. Portion of a fusion weld that has been completely melted during welding. Weld metal has elements of filler metal and base metal melted in the weld thermal cycle.

Weld root. See *root of joint*.

Y-connection. HSS connection in which the *branch member* or connecting element is not perpendicular to the *main member* and in which forces transverse to the main member are primarily equilibriated by shear in the main member.

Yield moment. In a member subjected to bending, the moment at which the extreme outer fiber first attains the *yield stress.*

Yield point†. First stress in a material at which an increase in strain occurs without an increase in stress as defined by ASTM.

Yield strength†. *Stress* at which a material exhibits a specified limiting deviation from the proportionality of stress to strain as defined by ASTM.

Yield stress†. Generic term to denote either *yield point* or *yield strength*, as appropriate for the material.

Yielding. Limit state of inelastic deformation that occurs after the *yield stress* is reached.

Yielding (plastic moment). Yielding throughout the cross section of a member as the bending moment reaches the *plastic moment.*

Yielding (yield moment). Yielding at the extreme fiber on the cross section of a member when the bending moment reaches the *yield moment.*

CHAPTER A

GENERAL PROVISIONS

This chapter states the scope of the Specification, summarizes referenced specification, code, and standard documents, and provides requirements for materials and contract documents.

The chapter is organized as follows:

A1. Scope
A2. Referenced Specifications, Codes and Standards
A3. Material
A4. Structural Design Drawings and Specifications

A1. SCOPE

The *Specification for Structural Steel Buildings*, hereafter referred to as the Specification, shall apply to the design of the *structural steel* system, where the steel elements are defined in the AISC *Code of Standard Practice for Steel Buildings and Bridges*, Section 2.1.

This Specification includes the Symbols, the Glossary, Chapters A through M, and Appendices 1 through 7. The Commentary and the User Notes interspersed throughout are not part of the Specification.

User Note: User notes are intended to provide concise and practical guidance in the application of the provisions.

This Specification sets forth criteria for the design, fabrication, and erection of *structural steel* buildings and other structures, where other structures are defined as those structures designed, fabricated, and erected in a manner similar to buildings, with building-like vertical and lateral load resisting elements. Where conditions are not covered by the Specification, designs are permitted to be based on tests or analysis, subject to the approval of the *authority having jurisdiction*. Alternate methods of analysis and design shall be permitted, provided such alternate methods or criteria are acceptable to the authority having jurisdiction.

User Note: For the design of structural members, other than hollow structural sections (*HSS*), that are cold-formed to shapes, with elements not more than 1 in. (25 mm) in thickness, the provisions in the AISI *North American Specification for the Design of Cold-Formed Steel Structural Members* are recommended.

1. Low-Seismic Applications

When the *seismic response modification coefficient*, *R*, (as specified in the *applicable building code*) is taken equal to or less than 3, the design, fabrication, and erection of structural-steel-framed buildings and other structures shall comply with this Specification.

2. High-Seismic Applications

When the *seismic response modification coefficient*, *R*, (as specified in the *applicable building code*) is taken greater than 3, the design, fabrication and erection of structural-steel-framed buildings and other structures shall comply with the requirements in the *Seismic Provisions for Structural Steel Buildings* (ANSI/AISC 341), in addition to the provisions of this Specification.

3. Nuclear Applications

The design of nuclear structures shall comply with the requirements of the *Specification for the Design, Fabrication, and Erection of Steel Safety-Related Structures in Nuclear Facilities* (ANSI/AISC N690) including Supplement No. 2 or the *Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities* (ANSI/AISC N690L), in addition to the provisions of this Specification.

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

The following specifications, codes and standards are referenced in this Specification:

ACI International (ACI)

ACI 318-02 *Building Code Requirements for Structural Concrete and Commentary*

ACI 318M-02 *Metric Building Code Requirements for Structural Concrete and Commentary*

American Institute of Steel Construction, Inc. (AISC)

AISC 303-05 *Code of Standard Practice for Steel Buildings and Bridges*

ANSI/AISC 341-05 *Seismic Provisions for Structural Steel Buildings*

ANSI/AISC N690-1994(R2004) *Specification for the Design, Fabrication and Erection of Steel Safety-Related Structures for Nuclear Facilities,* including Supplement No. 2

ANSI/AISC N690L-03 *Load and Resistance Factor Design Specification for Steel Safety-Related Structures for Nuclear Facilities*

American Society of Civil Engineers (ASCE)

SEI/ASCE 7-02 *Minimum Design Loads for Buildings and Other Structures*

ASCE/SFPE 29-99 *Standard Calculation Methods for Structural Fire Protection*

American Society of Mechanical Engineers (ASME)

ASME B18.2.6-96 *Fasteners for Use in Structural Applications*

ASME B46.1-95 *Surface Texture, Surface Roughness, Waviness, and Lay*

ASTM International (ASTM)

A6/A6M-04a *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling*

A36/A36M-04 *Standard Specification for Carbon Structural Steel*

A53/A53M-02 *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*

A193/A193M-04a *Standard Specification for Alloy-Steel and Stainless Steel Bolting Materials for High-Temperature Service*

A194/A194M-04 *Standard Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both*

A216/A216M-93(2003) *Standard Specification for Steel Castings, Carbon, Suitable for Fusion Welding, for High Temperature Service*

A242/A242M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel*

A283/A283M-03 *Standard Specification for Low and Intermediate Tensile Strength Carbon Steel Plates*

A307-03 *Standard Specification for Carbon Steel Bolts and Studs, 60,000 PSI Tensile Strength*

A325-04 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

A325M-04 *Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)*

A354-03a *Standard Specification for Quenched and Tempered Alloy Steel Bolts, Studs, and Other Externally Threaded Fasteners*

A370-03a *Standard Test Methods and Definitions for Mechanical Testing of Steel Products*

A449-04 *Standard Specification for Quenched and Tempered Steel Bolts and Studs*

A490-04 *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

A490M-04 *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)*

A500-03a *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*

A501-01 *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*

A502-03 *Standard Specification for Steel Structural Rivets*

A514/A514M-00a *Standard Specification for High-Yield Strength, Quenched and Tempered Alloy Steel Plate, Suitable for Welding*

A529/A529M-04 *Standard Specification for High-Strength Carbon-Manganese Steel of Structural Quality*

A563-04 *Standard Specification for Carbon and Alloy Steel Nuts*

A563M-03 *Standard Specification for Carbon and Alloy Steel Nuts [Metric]*

A568/A568M-03 *Standard Specification for Steel, Sheet, Carbon, and High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, General Requirements for*

A572/A572M-04 *Standard Specification for High-Strength Low-Alloy Columbium-Vanadium Structural Steel*

A588/A588M-04 *Standard Specification for High-Strength Low-Alloy Structural Steel with 50 ksi [345 MPa] Minimum Yield Point to 4 in. [100 mm] Thick*

A606-04 *Standard Specification for Steel, Sheet and Strip, High-Strength, Low-Alloy, Hot-Rolled and Cold-Rolled, with Improved Atmospheric Corrosion Resistance*

A618/A618M-04 *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*

A673/A673M-04 *Standard Specification for Sampling Procedure for Impact Testing of Structural Steel*

A668/A668M-04 *Standard Specification for Steel Forgings, Carbon and Alloy, for General Industrial Use*

A709/A709M-04 *Standard Specification for Carbon and High-Strength Low-Alloy Structural Steel Shapes, Plates, and Bars and Quenched-and-Tempered Alloy Structural Steel Plates for Bridges*

A751-01 *Standard Test Methods, Practices, and Terminology for Chemical Analysis of Steel Products*

A847-99a(2003) *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*

A852/A852M-03 *Standard Specification for Quenched and Tempered Low-Alloy Structural Steel Plate with 70 ksi [485 MPa] Minimum Yield Strength to 4 in. [100 mm] Thick*

A913/A913M-04 *Standard Specification for High-Strength Low-Alloy Steel Shapes of Structural Quality, Produced by Quenching and Self-Tempering Process (QST)*

A992/A992M-04 *Standard Specification for Steel for Structural Shapes for Use in Building Framing*

User Note: ASTM A992 is the most commonly referenced specification for W shapes.

A1011/A1011M-04 *Standard Specification for Steel, Sheet and Strip, Hot-Rolled, Carbon, Structural, High-Strength Low-Alloy and High-Strength Low-Alloy with Improved Formability*

C33-03 *Standard Specification for Concrete Aggregates*

C330-04 *Standard Specification for Lightweight Aggregates for Structural Concrete*

E119-00a *Standard Test Methods for Fire Tests of Building Construction and Materials*

E709-01 *Standard Guide for Magnetic Particle Examination*

F436-03 *Standard Specification for Hardened Steel Washers*

F959-02 *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

F1554-99 *Standard Specification for Anchor Bolts, Steel, 36, 55, and 105 ksi Yield Strength*

> **User Note:** ASTM F1554 is the most commonly referenced specification for anchor rods. Grade and weldability must be specified.

F1852-04 *Standard Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

<u>American Welding Society (AWS)</u>

AWS D1.1/D1.1M-2004 *Structural Welding Code–Steel*

AWS A5.1-2004 *Specification for Carbon Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.5-96 *Specification for Low-Alloy Steel Electrodes for Shielded Metal Arc Welding*

AWS A5.17/A5.17M-97 *Specification for Carbon Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.18:2001 *Specification for Carbon Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.20-95 *Specification for Carbon Steel Electrodes for Flux Cored Arc Welding*

AWS A5.23/A5.23M-97 *Specification for Low-Alloy Steel Electrodes and Fluxes for Submerged Arc Welding*

AWS A5.25/A5.25M-97 *Specification for Carbon and Low-Alloy Steel Electrodes and Fluxes for Electroslag Welding*

AWS A5.26/A5.26M-97 *Specification for Carbon and Low-Alloy Steel Electrodes for Electrogas Welding*

AWS A5.28-96 *Specification for Low-Alloy Steel Electrodes and Rods for Gas Shielded Arc Welding*

AWS A5.29:1998 *Specification for Low-Alloy Steel Electrodes for Flux Cored Arc Welding*

<u>Research Council on Structural Connections (RCSC)</u>

Specification for Structural Joints Using ASTM A325 or A490 Bolts, 2004

A3. MATERIAL

1. Structural Steel Materials

Material test reports or reports of tests made by the fabricator or a testing laboratory shall constitute sufficient evidence of conformity with one of the above listed ASTM standards. For hot-rolled structural shapes, plates, and bars, such tests shall be made in accordance with ASTM A6/A6M; for sheets, such tests shall be made in accordance with ASTM A568/A568M; for *tubing* and *pipe*, such tests shall be made in accordance with the requirements of the applicable ASTM standards listed above for those product forms. If requested, the fabricator shall provide an affidavit stating that the *structural steel* furnished meets the requirements of the grade specified.

1a. ASTM Designations

Structural steel material conforming to one of the following ASTM specifications is approved for use under this Specification:

(1) Hot-rolled structural shapes

ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M
ASTM A588/A588M
ASTM A709/A709M
ASTM A913/A913M
ASTM A992/ A992M

(2) Structural tubing

ASTM A500
ASTM A501
ASTM A618
ASTM A847

(3) Pipe

ASTM A53/A53M, Gr. B

(4) Plates

ASTM A36/A36M
ASTM A242/A242M
ASTM A283/A283M
ASTM A514/A514M
ASTM A529/A529M
ASTM A572/A572M
ASTM A588/A588M
ASTM A709/A709M
ASTM A852/A852M
ASTM A1011/A1011M

(5) Bars

ASTM A36/A36M
ASTM A529/A529M
ASTM A572/A572M
ASTM A709/A709M

(6) Sheets

ASTM A606
A1011/A1011M SS, HSLAS, AND HSLAS-F

1b. Unidentified Steel

Unidentified steel free of injurious defects is permitted to be used for unimportant members or details, where the precise physical properties and weldability of the steel would not affect the strength of the structure.

1c. Rolled Heavy Shapes

ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm), used as members subject to primary (computed) tensile *force*s due to tension or flexure and spliced using *complete-joint-penetration groove weld*s that fuse through the thickness of the member, shall be specified as follows. The contract documents shall require that such shapes be supplied with *Charpy V-Notch (CVN) impact test* results in accordance with ASTM A6/A6M, Supplementary Requirement S30, *Charpy V-Notch Impact Test for Structural Shapes – Alternate Core Location*. The impact test shall meet a minimum average value of 20 ft-lbs (27 J) absorbed energy at +70 °F (+21 °C).

The above requirements do not apply if the *splice*s and *connection*s are made by bolting. The above requirements do not apply to hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) that have shapes with flange or web elements less than 2 in. (50 mm) thick welded with *complete-joint-penetration groove weld*s to the face of the shapes with thicker elements.

> **User Note:** Additional requirements for joints in heavy rolled members are given in Sections J1.5, J1.6, J2.7, and M2.2.

1d. Built-Up Heavy Shapes

Built-up cross-sections consisting of plates with a thickness exceeding 2 in. (50 mm), used as members subject to primary (computed) tensile *force*s due to tension or flexure and spliced or connected to other members using *complete-joint-penetration groove weld*s that fuse through the thickness of the plates, shall be specified as follows. The contract documents shall require that the steel be supplied with *Charpy V-Notch impact test* results in accordance with ASTM A6/A6M, Supplementary Requirement S5, *Charpy V-Notch Impact Test.* The impact test shall be conducted in accordance with ASTM A673/A673M, Frequency P, and shall meet a minimum average value of 20 ft-lbs (27 J) absorbed energy at +70 °F (+21 °C).

The above requirements also apply to built-up cross-sections consisting of plates exceeding 2 in. (50 mm) that are welded with complete-joint-penetration groove welds to the face of other sections.

> **User Note:** Additional requirements for joints in heavy built-up members are given in Sections J1.5, J1.6, J2.7, and M2.2.

2. Steel Castings and Forgings

Cast steel shall conform to ASTM A216/A216M, Gr. WCB with Supplementary Requirement S11. Steel forgings shall conform to ASTM A668/A668M. Test reports produced in accordance with the above reference standards shall constitute sufficient evidence of conformity with such standards.

3. Bolts, Washers and Nuts

Bolt, washer, and nut material conforming to one of the following ASTM specifications is approved for use under this Specification:

(1) Bolts:

ASTM A307
ASTM A325
ASTM A325M
ASTM A449
ASTM A490
ASTM A490M
ASTM F1852

(2) Nuts:

ASTM A194/A194M
ASTM A563
ASTM A563M

(3) Washers:

ASTM F436
ASTM F436M

(4) Compressible-Washer-Type Direct Tension Indicators:

ASTM F959
ASTM F959M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

4. Anchor Rods and Threaded Rods

Anchor rod and threaded rod material conforming to one of the following ASTM specifications is approved for use under this Specification:

ASTM A36/A36M
ASTM A193/A193M
ASTM A354
ASTM A449
ASTM A572/A572M
ASTM A588/A588M
ASTM F1554

User Note: ASTM F1554 is the preferred material specification for anchor rods.

A449 material is acceptable for high-strength anchor rods and threaded rods of any diameter.

Threads on anchor rods and threaded rods shall conform to the Unified Standard Series of ASME B18.2.6 and shall have Class 2A tolerances.

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards.

5. Filler Metal and Flux for Welding

Filler metals and fluxes shall conform to one of the following specifications of the American Welding Society:

AWS A5.1
AWS A5.5
AWS A5.17/A5.17M
AWS A5.18
AWS A5.20
AWS A5.23/A5.23M
AWS A5.25/A5.25M
AWS A5.26/A5.26M
AWS A5.28
AWS A5.29
AWS A5.32/A5.32M

Manufacturer's certification shall constitute sufficient evidence of conformity with the standards. Filler metals and fluxes that are suitable for the intended application shall be selected.

6. Stud Shear Connectors

Steel stud *shear connector*s shall conform to the requirements of *Structural Welding Code–Steel*, AWS D1.1.

> **User Note:** Studs are made from cold drawn bar, either semi-killed or killed aluminum or silicon deoxidized, conforming to the requirements of ASTM A29/A29M-04, Standard Specification for Steel Bars, Carbon and Alloy, Hot-Wrought, General Requirements for.

Manufacturer's certification shall constitute sufficient evidence of conformity with AWS D1.1.

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

The design drawings and specifications shall meet the requirements in the *Code of Standard Practice for Steel Buildings and Bridges*, except for deviations specifically identified in the design drawings and/or specifications.

CHAPTER B

DESIGN REQUIREMENTS

The general requirements for the analysis and design of steel structures that are applicable to all chapters of the specification are given in this chapter.

The chapter is organized as follows:

B1. General Provisions
B2. Loads and Load Combinations
B3. Design Basis
B4. Classification of Sections for Local Buckling
B5. Fabrication, Erection and Quality Control
B6. Evaluation of Existing Structures

B1. GENERAL PROVISIONS

The design of members and *connection*s shall be consistent with the intended behavior of the framing system and the assumptions made in the *structural analysis*. Unless restricted by the *applicable building code*, *lateral load* resistance and *stability* may be provided by any combination of members and connections.

B2. LOADS AND LOAD COMBINATIONS

The loads and load combinations shall be as stipulated by the *applicable building code*. In the absence of a building code, the loads and load combinations shall be those stipulated in SEI/ASCE 7. For design purposes, the *nominal load*s shall be taken as the *load*s stipulated by the applicable building code.

User Note: For LRFD designs, the load combinations in SEI/ASCE 7, Section 2.3 apply. For ASD designs, the load combinations in SEI/ASCE 7, Section 2.4 apply.

B3. DESIGN BASIS

Designs shall be made according to the provisions for *Load and Resistance Factor Design* (LRFD) or to the provisions for *Allowable Strength Design* (ASD).

1. Required Strength

The *required strength* of structural members and *connection*s shall be determined by *structural analysis* for the appropriate load combinations as stipulated in Section B2.

Design by *elastic*, *inelastic* or *plastic analysis* is permitted. Provisions for inelastic and plastic analysis are as stipulated in Appendix 1, Inelastic Analysis and Design. The provisions for moment redistribution in continuous beams in Appendix 1, Section 1.3 are permitted for elastic analysis only.

2. Limit States

Design shall be based on the principle that no applicable strength or serviceability *limit state* shall be exceeded when the structure is subjected to all appropriate load combinations.

3. Design for Strength Using Load and Resistance Factor Design (LRFD)

Design according to the provisions for *Load and Resistance Factor Design* (LRFD) satisfies the requirements of this Specification when the *design strength* of each *structural component* equals or exceeds the *required strength* determined on the basis of the *LRFD load combinations*. All provisions of this Specification, except for those in Section B3.4, shall apply.

Design shall be performed in accordance with Equation B3-1:

$$R_u \leq \phi R_n \tag{B3-1}$$

where

R_u = required strength (LRFD)
R_n = *nominal strength*, specified in Chapters B through K
ϕ = *resistance factor*, specified in Chapters B through K
ϕR_n = design strength

4. Design for Strength Using Allowable Strength Design (ASD)

Design according to the provisions for *Allowable Strength Design* (ASD) satisfies the requirements of this Specification when the *allowable strength* of each *structural component* equals or exceeds the *required strength* determined on the basis of the *ASD load combinations*. All provisions of this Specification, except those of Section B3.3, shall apply.

Design shall be performed in accordance with Equation B3-2:

$$R_a \leq R_n / \Omega \tag{B3-2}$$

where

R_a = required strength (ASD)
R_n = *nominal strength*, specified in Chapters B through K
Ω = safety factor, specified in Chapters B through K
R_n/Ω = allowable strength

5. Design for Stability

Stability of the structure and its elements shall be determined in accordance with Chapter C.

6. Design of Connections

Connection elements shall be designed in accordance with the provisions of Chapters J and K. The *forces* and deformations used in design shall be consistent with the intended performance of the connection and the assumptions used in the *structural analysis*.

> **User Note:** Section 3.1.2 of the *Code of Standard Practice* addresses communication of necessary information for the design of connections.

6a. Simple Connections

A simple connection transmits a negligible moment across the connection. In the analysis of the structure, simple connections may be assumed to allow unrestrained relative rotation between the framing elements being connected. A simple connection shall have sufficient rotation capacity to accommodate the required rotation determined by the analysis of the structure. Inelastic rotation of the connection is permitted.

6b. Moment Connections

A moment connection transmits moment across the connection. Two types of moment connections, FR and PR, are permitted, as specified below.

(a) *Fully-Restrained (FR) Moment Connections*
A fully-restrained (FR) moment connection transfers moment with a negligible rotation between the connected members. In the analysis of the structure, the connection may be assumed to allow no relative rotation. An FR connection shall have sufficient strength and stiffness to maintain the angle between the connected members at the strength limit states.

(b) *Partially-Restrained (PR) Moment Connections*
Partially-restrained (PR) moment connections transfer moments, but the rotation between connected members is not negligible. In the analysis of the structure, the force-deformation response characteristics of the connection shall be included. The response characteristics of a PR connection shall be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection shall have sufficient strength, stiffness, and deformation capacity at the strength limit states.

7. Design for Serviceability

The overall structure and the individual members, *connection*s, and connectors shall be checked for serviceability. Performance requirements for serviceability design are given in Chapter L.

8. Design for Ponding

The roof system shall be investigated through *structural analysis* to assure adequate strength and *stability* under *ponding* conditions, unless the roof surface is provided with a slope of $^1/_4$ in. per ft (20 mm per meter) or greater toward points of free drainage or an adequate system of drainage is provided to prevent the accumulation of water.

See Appendix 2, Design for Ponding, for methods of checking ponding.

9. Design for Fatigue

Fatigue shall be considered in accordance with Appendix 3, Design for Fatigue, for members and their *connection*s subject to repeated *loading*. Fatigue need not be considered for seismic effects or for the effects of wind loading on normal building *lateral load resisting systems* and building enclosure components.

10. Design for Fire Conditions

Two methods of design for fire conditions are provided in Appendix 4, Structural Design for Fire Conditions: Qualification Testing and Engineering Analysis. Compliance with the fire protection requirements in the *applicable building code* shall be deemed to satisfy the requirements of this section and Appendix 4.

Nothing in this section is intended to create or imply a contractual requirement for the *engineer of record* responsible for the structural design or any other member of the design team.

User Note: Design by qualification testing is the prescriptive method specified in most building codes. Traditionally, on most projects where the architect is the prime professional, the architect has been the responsible party to specify and coordinate fire protection requirements. Design by Engineering Analysis is a new engineering approach to fire protection. Designation of the person(s) responsible for designing for fire conditions is a contractual matter to be addressed on each project.

11. Design for Corrosion Effects

Where corrosion may impair the strength or serviceability of a structure, *structural components* shall be designed to tolerate corrosion or shall be protected against corrosion.

12. Design Wall Thickness for HSS

The *design wall thickness*, t, shall be used in calculations involving the wall thickness of hollow structural sections (*HSS*). The design wall thickness, t, shall be taken equal to 0.93 times the nominal wall thickness for electric-resistance-welded (ERW) HSS and equal to the nominal thickness for submerged-arc-welded (SAW) HSS.

13. Gross and Net Area Determination

a. Gross Area

The gross area, A_g, of a member is the total cross-sectional area.

b. Net Area

The *net area*, A_n, of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as $^1/_{16}$ in. (2 mm) greater than the *nominal dimension* of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, of all holes in the chain, and adding, for each *gage* space in the chain, the quantity $s^2/4g$

where

s = longitudinal center-to-center spacing (*pitch*) of any two consecutive holes, in. (mm)

g = transverse center-to-center spacing (gage) between *fastener* gage lines, in. (mm)

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted *HSS* welded to a *gusset plate*, the net area, A_n, is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or *slot weld*s, the *weld metal* shall not be considered as adding to the net area.

User Note: Section J4.1(b) limits A_n to a maximum of $0.85A_g$ for splice plates with holes.

B4. CLASSIFICATION OF SECTIONS FOR LOCAL BUCKLING

Sections are classified as *compact*, *noncompact*, or *slender-element section*s. For a section to qualify as compact its flanges must be continuously connected to the web or webs and the width-thickness ratios of its compression elements must not exceed the limiting width-thickness ratios λ_p from Table B4.1. If the width-thickness ratio of one or more compression elements exceeds λ_p, but does not exceed λ_r from Table B4.1, the section is noncompact. If the width-thickness ratio of any element exceeds λ_r, the section is referred to as a *slender-element section*.

1. Unstiffened Elements

For *unstiffened element*s supported along only one edge parallel to the direction of the compression *force*, the width shall be taken as follows:

(a) For flanges of I-shaped members and tees, the width b is one-half the full-flange width, b_f.
(b) For legs of angles and flanges of channels and zees, the width b is the full *nominal dimension*.
(c) For plates, the width b is the distance from the free edge to the first row of *fastener*s or line of welds.
(d) For stems of tees, d is taken as the full nominal depth of the section.

User Note: Refer to Table B4.1 for the graphic representation of unstiffened element dimensions.

2. Stiffened Elements

For *stiffened element*s supported along two edges parallel to the direction of the compression *force*, the width shall be taken as follows:

(a) For webs of rolled or *formed section*s, h is the clear distance between flanges less the fillet or corner radius at each flange; h_c is twice the distance from the centroid to the inside face of the compression flange less the fillet or corner radius.
(b) For webs of built-up sections, h is the distance between adjacent lines of *fasteners* or the clear distance between flanges when welds are used, and h_c is twice the distance from the centroid to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used; h_p is twice the distance from the plastic neutral axis to the nearest line of fasteners at the compression flange or the inside face of the compression flange when welds are used.
(c) For flange or *diaphragm plate*s in built-up sections, the width b is the distance between adjacent lines of fasteners or lines of welds.
(d) For flanges of rectangular hollow structural sections (*HSS*), the width b is the clear distance between webs less the inside corner radius on each side. For webs of rectangular HSS, h is the clear distance between the flanges less the inside corner radius on each side. If the corner radius is not known, b and h shall be taken as the corresponding outside dimension minus three times the thickness. The thickness, t, shall be taken as the *design wall thickness*, per Section B3.12.

User Note: Refer to Table B4.1 for the graphic representation of stiffened element dimensions.

For tapered flanges of rolled sections, the thickness is the nominal value halfway between the free edge and the corresponding face of the web.

TABLE B4.1
Limiting Width-Thickness Ratios for Compression Elements

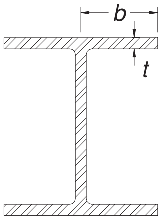
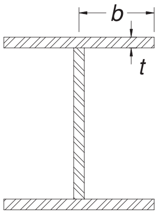
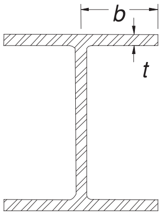
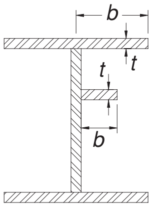
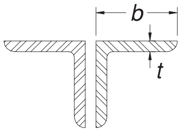
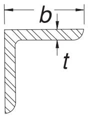
	Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios		Example
				λ_p (compact)	λ_r (noncompact)	
Unstiffened Elements	1	Flexure in flanges of rolled I-shaped sections and channels	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	2	Flexure in flanges of doubly and singly symmetric I-shaped built-up sections	b/t	$0.38\sqrt{E/F_y}$	$0.95\sqrt{k_cE/F_L}$ [a],[b]	
	3	Uniform compression in flanges of rolled I-shaped sections, plates projecting from rolled I-shaped sections; outstanding legs of pairs of angles in continuous contact and flanges of channels	b/t	NA	$0.56\sqrt{E/F_y}$	
	4	Uniform compression in flanges of built-up I-shaped sections and plates or angle legs projecting from built-up I-shaped sections	b/t	NA	$0.64\sqrt{k_cE/F_y}$ [a]	
	5	Uniform compression in legs of single angles, legs of double angles with separators, and all other unstiffened elements	b/t	NA	$0.45\sqrt{E/F_y}$	
	6	Flexure in legs of single angles	b/t	$0.54\sqrt{E/F_y}$	$0.91\sqrt{E/F_y}$	

TABLE B4.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

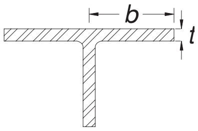
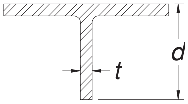
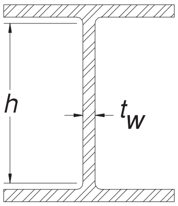
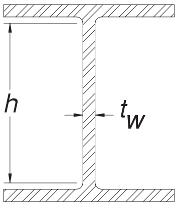
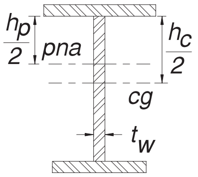
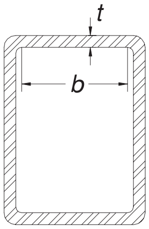
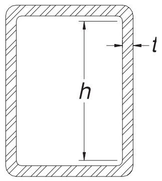
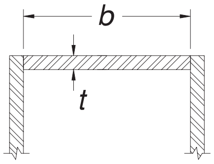
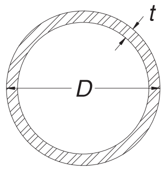
	Case	Description of Element	Width Thickness Ratio	Limiting Width-Thickness Ratios		Example
				λ_p (compact)	λ_r (noncompact)	
	7	Flexure in flanges of tees	b/t	$0.38\sqrt{E/F_y}$	$1.0\sqrt{E/F_y}$	
	8	Uniform compression in stems of tees	d/t	NA	$0.75\sqrt{E/F_y}$	
Stiffened Elements	**9**	Flexure in webs of doubly symmetric I-shaped sections and channels	h/t_w	$3.76\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	
	10	Uniform compression in webs of doubly symmetric I-shaped sections	h/t_w	NA	$1.49\sqrt{E/F_y}$	
	11	Flexure in webs of singly-symmetric I-shaped sections	h_c/t_w	$\dfrac{\dfrac{h_c}{h_p}\sqrt{\dfrac{E}{F_y}}}{\left(0.54\dfrac{M_p}{M_y}-0.09\right)^2} \le \lambda_r$	$5.70\sqrt{E/F_y}$	
	12	Uniform compression in flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression; flange cover plates and diaphragm plates between lines of fasteners or welds	b/t	$1.12\sqrt{E/F_y}$	$1.40\sqrt{E/F_y}$	
	13	Flexure in webs of rectangular HSS	h/t	$2.42\sqrt{E/F_y}$	$5.70\sqrt{E/F_y}$	

TABLE B4.1 (cont.)
Limiting Width-Thickness Ratios for Compression Elements

Case	Description of Element	Width Thick-ness Ratio	Limiting Width-Thickness Ratios		Example
			λ_p (compact)	λ_r (noncompact)	
14	Uniform compression in all other stiffened elements	b/t	NA	$1.49\sqrt{E/F_y}$	
15	Circular hollow sections				
	In uniform compression	D/t	NA	$0.11\,E/F_y$	
	In flexure	D/t	$0.07\,E/F_y$	$0.31\,E/F_y$	

[a] $k_c = \frac{4}{\sqrt{h/t_w}}$, but shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes. (See Cases 2 and 4)

[b] $F_L = 0.7F_y$ for minor-axis bending, major axis bending of slender-web built-up I-shaped members, and major axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} \geq 0.7$; $F_L = F_y S_{xt}/S_{xc} \geq 0.5F_y$ for major-axis bending of compact and noncompact web built-up I-shaped members with $S_{xt}/S_{xc} < 0.7$. (See Case 2)

B5. FABRICATION, ERECTION AND QUALITY CONTROL

Shop drawings, fabrication, shop painting, erection, and *quality control* shall meet the requirements stipulated in Chapter M, Fabrication, Erection, and Quality Control.

B6. EVALUATION OF EXISTING STRUCTURES

Provisions for the evaluation of existing structures are presented in Appendix 5, Evaluation of Existing Structures.

CHAPTER C

STABILITY ANALYSIS AND DESIGN

This chapter addresses general requirements for the stability analysis and design of members and frames.

The chapter is organized as follows:

C1. Stability Design Requirements
C2. Calculation of Required Strengths

C1. STABILITY DESIGN REQUIREMENTS

1. General Requirements

Stability shall be provided for the structure as a whole and for each of its elements. Any method that considers the influence of *second-order effects* (including P-Δ and P-δ *effects*), flexural, shear and axial deformations, geometric imperfections, and member stiffness reduction due to residual stresses on the stability of the structure and its elements is permitted. The methods prescribed in this chapter and Appendix 7, Direct Analysis Method, satisfy these requirements. All component and *connection* deformations that contribute to the lateral displacements shall be considered in the stability analysis.

In structures designed by elastic analysis, individual member stability and stability of the structure as a whole are provided jointly by:

(1) Calculation of the *required strengths* for members, connections and other elements using one of the methods specified in Section C2.2, and
(2) Satisfaction of the member and connection design requirements in this specification based upon those required strengths.

In structures designed by inelastic analysis, the provisions of Appendix 1, Inelastic Analysis and Design, shall be satisfied.

2. Member Stability Design Requirements

Individual member stability is provided by satisfying the provisions of Chapters E, F, G, H and I.

User Note: Local buckling of cross section components can be avoided by the use of compact sections defined in Section B4.

Where elements are designed to function as braces to define the unbraced length of columns and beams, the bracing system shall have sufficient stiffness and strength to control member movement at the braced points. Methods of satisfying this requirement are provided in Appendix 6, Stability Bracing for Columns and Beams.

3. System Stability Design Requirements

Lateral stability shall be provided by *moment frames*, *braced frames*, *shear walls*, and/or other equivalent *lateral load resisting systems*. The overturning effects of *drift* and the destabilizing influence of *gravity loads* shall be considered. *Force* transfer and *load* sharing between elements of the framing systems shall be considered. Braced-frame and shear-wall systems, moment frames, gravity framing systems, and *combined systems* shall satisfy the following specific requirements:

3a. Braced-Frame and Shear-Wall Systems

In structures where lateral stability is provided solely by diagonal bracing, shear walls, or equivalent means, the *effective length factor*, K, for compression members shall be taken as 1.0, unless *structural analysis* indicates that a smaller value is appropriate. In braced-frame systems, it is permitted to design the columns, beams, and diagonal members as a vertically cantilevered, simply connected truss.

User Note: Knee-braced frames function as moment-frame systems and should be treated as indicated in Section C1.3b. Eccentrically braced frame systems function as combined systems and should be treated as indicated in Section C1.3d.

3b. Moment-Frame Systems

In frames where lateral stability is provided by the flexural stiffness of connected beams and columns, the effective length factor K or elastic critical buckling stress, F_e, for *columns* and *beam-columns* shall be determined as specified in Section C2.

3c. Gravity Framing Systems

Columns in gravity framing systems shall be designed based on their actual length ($K = 1.0$) unless analysis shows that a smaller value may be used. The lateral stability of gravity framing systems shall be provided by moment frames, braced frames, shear walls, and/or other equivalent lateral load resisting systems. *P-Δ effects* due to load on the gravity columns shall be transferred to the lateral load resisting systems and shall be considered in the calculation of the required strengths of the lateral load resisting systems.

3d. Combined Systems

The analysis and design of members, *connections* and other elements in combined systems of moment frames, braced frames, and/or shear walls and gravity frames shall meet the requirements of their respective systems.

C2. CALCULATION OF REQUIRED STRENGTHS

Except as permitted in Section C2.2b, *required strengths* shall be determined using a *second-order analysis* as specified in Section C2.1. Design by either second-order or *first-order analysis* shall meet the requirements specified in Section C2.2.

1. Methods of Second-Order Analysis

Second-order analysis shall conform to the requirements in this Section.

1a. General Second-Order Elastic Analysis

Any second-order elastic analysis method that considers both P-Δ and P-δ *effects* may be used.

The Amplified First-Order Elastic Analysis Method defined in Section C2.1b is an accepted method for second-order elastic analysis of braced, moment, and combined framing systems.

1b. Second-Order Analysis by Amplified First-Order Elastic Analysis

User Note: A method is provided in this section to account for second-order effects in frames by amplifying the axial forces and moments in members and connections from a first-order analysis.

The following is an approximate second-order analysis procedure for calculating the required flexural and axial strengths in members of *lateral load resisting systems.* The required second-order flexural strength, M_r, and axial strength, P_r, shall be determined as follows:

$$M_r = B_1 M_{nt} + B_2 M_{lt} \qquad \text{(C2-1a)}$$

$$P_r = P_{nt} + B_2 P_{lt} \qquad \text{(C2-1b)}$$

where

$$B_1 = \frac{C_m}{1 - \alpha P_r / P_{e1}} \geq 1 \qquad \text{(C2-2)}$$

For members subjected to axial compression, B_1 may be calculated based on the first-order estimate $P_r = P_{nt} + P_{lt}$.

User Note: B_1 is an amplifier to account for second order effects caused by displacements between brace points (P-δ) and B_2 is an amplifier to account for second order effects caused by displacements of braced points (P-Δ).

For members in which $B_1 \leq 1.05$, it is conservative to amplify the sum of the non-sway and sway moments (as obtained, for instance, by a first-order elastic analysis) by the B_2 amplifier, in other words, $M_r = B_2(M_{nt} + M_{lt})$.

$$B_2 = \frac{1}{1 - \dfrac{\alpha \Sigma P_{nt}}{\Sigma P_{e2}}} \geq 1 \qquad \text{(C2-3)}$$

User Note: Note that the B_2 amplifier (Equation C2-3) can be estimated in preliminary design by using a maximum lateral drift limit corresponding to the story shear ΣH in Equation C2-6b.

and

$$\alpha = 1.00\,(\text{LRFD}) \qquad \alpha = 1.60\,(\text{ASD})$$

M_r = required second-order flexural strength using LRFD or ASD load combinations, kip-in. (N-mm)

M_{nt} = first-order moment using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, kip-in. (N-mm)

M_{lt} = first-order moment using LRFD or ASD load combinations caused by lateral translation of the frame only, kip-in. (N-mm)

P_r = required second-order axial strength using LRFD or ASD load combinations, kips (N)

P_{nt} = first-order axial force using LRFD or ASD load combinations, assuming there is no lateral translation of the frame, kips (N)

ΣP_{nt} = total vertical load supported by the story using LRFD or ASD load combinations, including gravity column loads, kips (N)

P_{lt} = first-order axial force using LRFD or ASD load combinations caused by lateral translation of the frame only, kips (N)

C_m = a coefficient assuming no lateral translation of the frame whose value shall be taken as follows:

(i) For beam-columns not subject to transverse loading between supports in the plane of bending,

$$C_m = 0.6 - 0.4(M_1/M_2) \tag{C2-4}$$

where M_1 and M_2, calculated from a first-order analysis, are the smaller and larger moments, respectively, at the ends of that portion of the member unbraced in the plane of bending under consideration. M_1/M_2 is positive when the member is bent in reverse curvature, negative when bent in single curvature.

(ii) For beam-columns subjected to transverse loading between supports, the value of C_m shall be determined either by analysis or conservatively taken as 1.0 for all cases.

P_{e1} = elastic critical buckling resistance of the member in the plane of bending, calculated based on the assumption of zero sidesway, kips (N)

$$P_{e1} = \frac{\pi^2 EI}{(K_1 L)^2} \tag{C2-5}$$

ΣP_{e2} = elastic critical buckling resistance for the story determined by sidesway buckling analysis, kips (N)

For moment frames, where sidesway buckling effective length factors K_2 are determined for the columns, it is permitted to calculate the elastic story sidesway buckling resistance as

$$\Sigma P_{e2} = \Sigma \frac{\pi^2 EI}{(K_2 L)^2} \tag{C2-6a}$$

For all types of lateral load resisting systems, it is permitted to use

$$\Sigma P_{e2} = R_M \frac{\Sigma HL}{\Delta_H} \tag{C2-6b}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

R_M = 1.0 for braced-frame systems;

= 0.85 for moment-frame and combined systems, unless a larger value is justified by analysis

I = moment of inertia in the plane of bending, in.^{4} (mm^4)

L = story height, in. (mm)

K_1 = effective length factor in the plane of bending, calculated based on the assumption of no lateral translation, set equal to 1.0 unless analysis indicates that a smaller value may be used

K_2 = effective length factor in the plane of bending, calculated based on a sidesway buckling analysis

User Note: Methods for calculation of K_2 are discussed in the Commentary.

Δ_H = first-order interstory drift due to lateral forces, in. (mm). Where Δ_H varies over the plan area of the structure, Δ_H shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

ΣH = story shear produced by the lateral forces used to compute Δ_H, kips (N)

2. Design Requirements

These requirements apply to all types of braced, moment, and combined framing systems. Where the ratio of second-order drift to first-order drift is equal to or less than 1.5, the *required strengths* of members, *connections* and other elements shall be determined by one of the methods specified in Sections C2.2a or C2.2b, or by the *Direct Analysis Method* of Appendix 7. Where the ratio of second-order drift to first-order drift is greater than 1.5, the required strengths shall be determined by the Direct Analysis Method of Appendix 7.

User Note: The ratio of second-order drift to first-order drift can be represented by B_2, as calculated using Equation C2-3. Alternatively, the ratio can be calculated by comparing the results of a second-order analysis to the results of a first-order analysis, where the analyses are conducted either under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

For the methods specified in Sections 2.2a or 2.2b:

(1) Analyses shall be conducted according to the design and loading requirements specified in either Section B3.3 (LRFD) or Section B3.4 (ASD).

(2) The structure shall be analyzed using the nominal geometry and the nominal elastic stiffness for all elements.

2a. Design by Second-Order Analysis

Where required strengths are determined by a *second-order analysis*:

(1) The provisions of Section C2.1 shall be satisfied.

(2) For design by ASD, analyses shall be carried out under 1.6 times the *ASD load combinations* and the results shall be divided by 1.6 to obtain the required strengths.

User Note: The amplified first order analysis method of Section C2.1b incorporates the 1.6 multiplier directly in the B_1 and B_2 amplifiers, such that no other modification is needed.

(3) All gravity-only load combinations shall include a minimum lateral load applied at each level of the structure of $0.002Y_i$, where Y_i is the *design gravity load* applied at level i, kips (N). This minimum *lateral load* shall be considered independently in two orthogonal directions.

User Note: The minimum lateral load of $0.002Y_i$, in conjunction with the other design-analysis constraints listed in this section, limits the error that would otherwise be caused by neglecting initial out-of-plumbness and member stiffness reduction due to residual stresses in the analysis.

(4) Where the ratio of second-order drift to first-order drift is less than or equal to 1.1, members are permitted to be designed using $K = 1.0$. Otherwise, *columns* and *beam-columns* in *moment frames* shall be designed using a K factor or column buckling stress, F_e, determined from a sidesway buckling analysis of the structure. Stiffness reduction adjustment due to column inelasticity is permitted in the determination of the K factor. For *braced frames*, K for compression members shall be taken as 1.0, unless structural analysis indicates a smaller value may be used.

2b. Design by First-Order Analysis

Required strengths are permitted to be determined by a first-order analysis, with all members designed using $K = 1.0$, provided that

(1) The required compressive strengths of all members whose flexural stiffnesses are considered to contribute to the lateral stability of the structure satisfy the following limitation:

$$\alpha P_r \le 0.5 P_y \tag{C2-7}$$

where

$$\alpha = 1.0 \text{ (LRFD)} \quad \alpha = 1.6 \text{ (ASD)}$$

P_r = required axial compressive strength under LRFD or ASD load combinations, kips (N)

P_y = member yield strength ($= AF_y$), kips (N)

(2) All load combinations include an additional lateral load, N_i, applied in combination with other loads at each level of the structure, where

$$N_i = 2.1(\Delta/L)Y_i \ge 0.0042Y_i \tag{C2-8}$$

Y_i = gravity load from the LRFD load combination or 1.6 times the ASD load combination applied at level i, kips (N)

Δ/L = the maximum ratio of Δ to L for all stories in the structure

Δ = first-order interstory drift due to the design loads, in. (mm). Where Δ varies over the plan area of the structure, Δ shall be the average drift weighted in proportion to vertical load or, alternatively, the maximum drift.

L = story height, in. (mm)

User Note: The drift Δ is calculated under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

This additional lateral load shall be considered independently in two orthogonal directions.

(3) The non-sway amplification of beam-column moments is considered by applying the B_1 amplifier of Section C2.1 to the total member moments.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

This chapter applies to members subject to axial tension caused by static *force*s acting through the centroidal axis.

The chapter is organized as follows:

D1. Slenderness Limitations
D2. Tensile Strength
D3. Area Determination
D4. Built-Up Members
D5. Pin-Connected Members
D6. Eyebars

User Note: For cases not included in this chapter the following sections apply:

• B3.9	Members subject to fatigue.
• Chapter H	Members subject to combined axial tension and flexure.
• J3.	Threaded rods.
• J4.1	Connecting elements in tension.
• J4.3	Block shear rupture strength at end connections of tension members.

D1. SLENDERNESS LIMITATIONS

There is no maximum slenderness limit for design of members in tension.

User Note: For members designed on the basis of tension, the slenderness ratio L/r preferably should not exceed 300. This suggestion does not apply to rods or hangers in tension.

D2. TENSILE STRENGTH

The *design tensile strength*, $\phi_t P_n$, and the *allowable tensile strength*, P_n/Ω_t, of tension members, shall be the lower value obtained according to the *limit state*s of *tensile yielding* in the gross section and *tensile rupture* in the net section.

(a) For tensile yielding in the gross section:

$$P_n = F_y A_g \tag{D2-1}$$

$$\phi_t = 0.90 \text{ (LRFD)} \qquad \Omega_t = 1.67 \text{ (ASD)}$$

(b) For tensile rupture in the net section:

$$P_n = F_u A_e \tag{D2-2}$$

$$\phi_t = 0.75 \text{ (LRFD)} \qquad \Omega_t = 2.00 \text{ (ASD)}$$

where

A_e = *effective net area*, in.^{2} (mm^2)
A_g = gross area of member, in.^{2} (mm^2)
F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
F_u = *specified minimum tensile strength* of the type of steel being used, ksi (MPa)

When members without holes are fully connected by welds, the effective net area used in Equation D2-2 shall be as defined in Section D3. When holes are present in a member with welded end *connection*s, or at the welded connection in the case of plug or *slot weld*s, the effective net area through the holes shall be used in Equation D2-2.

D3. AREA DETERMINATION

1. Gross Area

The gross area, A_g, of a member is the total cross-sectional area.

2. Net Area

The *net area*, A_n, of a member is the sum of the products of the thickness and the net width of each element computed as follows:

In computing net area for tension and shear, the width of a bolt hole shall be taken as $^1/_{16}$ in. (2 mm) greater than the *nominal dimension* of the hole.

For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the part shall be obtained by deducting from the gross width the sum of the diameters or slot dimensions as provided in Section J3.2, of all holes in the chain, and adding, for each *gage* space in the chain, the quantity $s^2/4g$

where

s = longitudinal center-to-center spacing (*pitch*) of any two consecutive holes, in. (mm)
g = transverse center-to-center spacing (gage) between *fastener* gage lines, in. (mm)

For angles, the gage for holes in opposite adjacent legs shall be the sum of the gages from the back of the angles less the thickness.

For slotted *HSS* welded to a *gusset plate*, the net area, A_n, is the gross area minus the product of the thickness and the total width of material that is removed to form the slot.

In determining the net area across plug or *slot welds*, the *weld metal* shall not be considered as adding to the net area.

User Note: Section J4.1(b) limits A_n to a maximum of $0.85A_g$ for splice plates with holes.

3. Effective Net Area

The effective area of tension members shall be determined as follows:

$$A_e = A_n U \tag{D3-1}$$

where U, the shear lag factor, is determined as shown in Table D3.1.

Members such as single angles, double angles and WT sections shall have *connections* proportioned such that U is equal to or greater than 0.60. Alternatively, a lesser value of U is permitted if these tension members are designed for the effect of eccentricity in accordance with H1.2 or H2.

D4. BUILT-UP MEMBERS

For limitations on the longitudinal spacing of connectors between elements in continuous contact consisting of a plate and a shape or two plates, see Section J3.5.

Either perforated *cover plates* or *tie plates* without *lacing* are permitted to be used on the open sides of built-up tension members. Tie plates shall have a length not less than two-thirds the distance between the lines of welds or *fasteners* connecting them to the components of the member. The thickness of such tie plates shall not be less than one-fiftieth of the distance between these lines. The longitudinal spacing of intermittent welds or fasteners at tie plates shall not exceed 6 in. (150 mm).

User Note: The longitudinal spacing of connectors between components should preferably limit the slenderness ratio in any component between the connectors to 300.

D5. PIN-CONNECTED MEMBERS

1. Tensile Strength

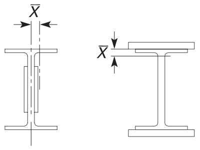
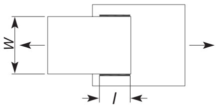
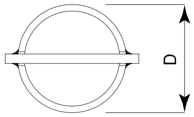
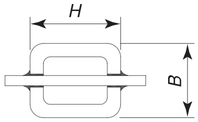
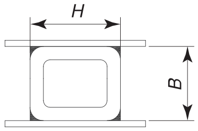
The *design tensile strength*, $\phi_t P_n$, and the *allowable tensile strength*, P_n/Ω_t, of pin-connected members, shall be the lower value obtained according to the *limit states* of *tensile rupture*, *shear rupture*, *bearing,* and *yielding*.

(a) For tensile rupture on the net effective area:

$$P_n = 2tb_{eff}F_u \tag{D5-1}$$

$$\phi_t = 0.75\,(\text{LRFD}) \qquad \Omega_t = 2.00\,(\text{ASD})$$

TABLE D3.1
Shear Lag Factors for Connections to Tension Members

Case	Description of Element		Shear Lag Factor, U	Example
1	All tension members where the tension load is transmitted directly to each of cross-sectional elements by fasteners or welds. (except as in Cases 3, 4, 5 and 6)		$U = 1.0$	——
2	All tension members, except plates and HSS, where the tension load is transmitted to some but not all of the cross-sectional elements by fasteners or longitudinal welds (Alternatively, for W, M, S and HP, Case 7 may be used.)		$U = 1 - \bar{x}/l$	
3	All tension members where the tension load is transmitted by transverse welds to some but not all of the cross-sectional elements.		$U = 1.0$ and $A_n =$ area of the directly connected elements	——
4	Plates where the tension load is transmitted by longitudinal welds only.		$l \geq 2w \ldots U = 1.0$ $2w > l \geq 1.5w \ldots U = 0.87$ $1.5w > l \geq w \ldots U = 0.75$	
5	Round HSS with a single concentric gusset plate		$l \geq 1.3D \ldots U = 1.0$ $D \leq l < 1.3D \ldots U = 1 - \bar{x}/l$ $\bar{x} = D/\pi$	
6	Rectangular HSS	with a single concentric gusset plate	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2 + 2BH}{4(B+H)}$	
		with two side gusset plates	$l \geq H \ldots U = 1 - \bar{x}/l$ $\bar{x} = \dfrac{B^2}{4(B+H)}$	
7	W, M, S or HP Shapes or Tees cut from these shapes. (If U is calculated per Case 2, the larger value is permitted to be used)	with flange connected with 3 or more fasteners per line in direction of loading	$b_f \geq 2/3d \ldots U = 0.90$ $b_f < 2/3d \ldots U = 0.85$	——
		with web connected with 4 or more fasteners in the direction of loading	$U = 0.70$	——
8	Single angles (If U is calculated per Case 2, the larger value is permitted to be used)	with 4 or more fasteners per line in direction of loading	$U = 0.80$	——
		with 2 or 3 fasteners per line in the direction of loading	$U = 0.60$	——

l = length of connection, in. (mm); w = plate width, in. (mm); $\bar{x}$ = connection eccentricity, in. (mm); B = overall width of rectangular HSS member, measured 90 degrees to the plane of the connection, in. (mm); H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)

(b) For shear rupture on the effective area:

$$P_n = 0.6F_u A_{sf} \tag{D5-2}$$

$$\phi_{sf} = 0.75\,(\text{LRFD}) \qquad \Omega_{sf} = 2.00\,(\text{ASD})$$

where

$A_{sf} = 2t(a + d/2)$, in.^{2}(mm^2)

a = shortest distance from edge of the pin hole to the edge of the member measured parallel to the direction of the *force*, in. (mm)

$b_{eff} = 2t + 0.63$, in. ($= 2t + 16$, mm) but not more than the actual distance from the edge of the hole to the edge of the part measured in the direction normal to the applied force

d = pin diameter, in. (mm)

t = thickness of plate, in. (mm)

(c) For bearing on the projected area of the pin, see Section J7.

(d) For yielding on the gross section, use Equation D2-1.

2. Dimensional Requirements

The pin hole shall be located midway between the edges of the member in the direction normal to the applied *force*. When the pin is expected to provide for relative movement between connected parts while under full *load*, the diameter of the pin hole shall not be more than $^1/_{32}$ in. (1 mm) greater than the diameter of the pin.

The width of the plate at the pin hole shall not be less than $2b_{eff} + d$ and the minimum extension, a, beyond the bearing end of the pin hole, parallel to the axis of the member, shall not be less than $1.33 \times b_{eff}$.

The corners beyond the pin hole are permitted to be cut at 45° to the axis of the member, provided the *net area* beyond the pin hole, on a plane perpendicular to the cut, is not less than that required beyond the pin hole parallel to the axis of the member.

D6. EYEBARS

1. Tensile Strength

The *available tensile strength* of *eyebar*s shall be determined in accordance with Section D2, with A_g taken as the cross-sectional area of the body.

For calculation purposes, the width of the body of the eyebars shall not exceed eight times its thickness.

2. Dimensional Requirements

*Eyebar*s shall be of uniform thickness, without reinforcement at the pin holes, and have circular heads with the periphery concentric with the pin hole.

The radius of transition between the circular head and the eyebar body shall not be less than the head diameter.

The pin diameter shall not be less than seven-eighths times the eyebar body width, and the pin hole diameter shall not be more than $^1/_{32}$ in. (1 mm) greater than the pin diameter.

For steels having F_y greater than 70 ksi (485 MPa), the hole diameter shall not exceed five times the plate thickness, and the width of the eyebar body shall be reduced accordingly.

A thickness of less than $^1/_2$ in. (13 mm) is permissible only if external nuts are provided to tighten pin plates and *filler* plates into snug contact. The width from the hole edge to the plate edge perpendicular to the direction of applied *load* shall be greater than two-thirds and, for the purpose of calculation, not more than three-fourths times the eyebar body width.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

This chapter addresses members subject to axial compression through the centroidal axis.

The chapter is organized as follows:

E1. General Provisions
E2. Slenderness Limitations and Effective Length
E3. Compressive Strength for Flexural Buckling of Members without Slender Elements
E4. Compressive Strength for Torsional and Flexural-Torsional Buckling of Members without Slender Elements
E5. Single Angle Compression Members
E6. Built-Up Members
E7. Members with Slender Elements

User Note: For members not included in this chapter the following sections apply:

- H1. – H3. Members subject to combined axial compression and flexure.
- H4. Members subject to axial compression and torsion.
- J4.4 Compressive strength of connecting elements.
- I2. Composite axial members.

E1. GENERAL PROVISIONS

The *design compressive strength*, $\phi_c P_n$, and the *allowable compressive strength*, P_n/Ω_c, are determined as follows:

The *nominal compressive strength*, P_n, shall be the lowest value obtained according to the *limit states* of *flexural buckling*, *torsional buckling* and *flexural-torsional buckling*.

(a) For doubly symmetric and singly symmetric members the limit state of flexural buckling is applicable.
(b) For singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up *columns*, the limit states of torsional or flexural-torsional buckling are also applicable.

$$\phi_c = 0.90\ \text{(LRFD)} \quad \Omega_c = 1.67\ \text{(ASD)}$$

E2. SLENDERNESS LIMITATIONS AND EFFECTIVE LENGTH

The effective length factor, K, for calculation of column slenderness, KL/r, shall be determined in accordance with Chapter C,

where

L = laterally unbraced length of the member, in. (mm)
r = governing radius of gyration, in. (mm)
K = the *effective length factor* determined in accordance with Section C2

User Note: For members designed on the basis of compression, the slenderness ratio KL/r preferably should not exceed 200.

E3. COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to compression members with *compact* and *noncompact sections*, as defined in Section B4, for uniformly compressed elements.

User Note: When the torsional unbraced length is larger than the lateral unbraced length, this section may control the design of wide flange and similarly shaped columns.

The *nominal compressive strength*, P_n, shall be determined based on the *limit state* of *flexural buckling*.

$$P_n = F_{cr} A_g \tag{E3-1}$$

The *flexural buckling stress*, F_{cr}, is determined as follows:

(a) When $\dfrac{KL}{r} \le 4.71\sqrt{\dfrac{E}{F_y}}$ (or $F_e \ge 0.44F_y$)

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] F_y \tag{E3-2}$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{F_y}}$ (or $F_e < 0.44F_y$)

$$F_{cr} = 0.877F_e \tag{E3-3}$$

where

F_e = elastic critical buckling stress determined according to Equation E3-4, Section E4, or the provisions of Section C2, as applicable, ksi (MPa)

$$F_e = \frac{\pi^2 E}{\left(\dfrac{KL}{r}\right)^2} \tag{E3-4}$$

User Note: The two equations for calculating the limits and applicability of Sections E3(a) and E3(b), one based on KL/r and one based on F_e, provide the same result.

E4. COMPRESSIVE STRENGTH FOR TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

This section applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up *column*s with *compact* and *noncompact sections*, as defined in Section B4 for uniformly compressed elements. These provisions are not required for single angles, which are covered in Section E5.

The *nominal compressive strength*, P_n, shall be determined based on the *limit state*s of *flexural-torsional* and *torsional buckling*, as follows:

$$P_n = F_{cr} A_g \tag{E4-1}$$

(a) For double-angle and tee-shaped compression members:

$$F_{cr} = \left(\frac{F_{cry} + F_{crz}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{cry}F_{crz}H}{\left(F_{cry} + F_{crz}\right)^2}}\right] \tag{E4-2}$$

where F_{cry} is taken as F_{cr} from Equation E3-2 or E3-3, for *flexural buckling* about the y-axis of symmetry and $\frac{KL}{r} = \frac{KL}{r_y}$, and

$$F_{crz} = \frac{GJ}{A_g \bar{r}_o^2} \tag{E4-3}$$

(b) For all other cases, F_{cr} shall be determined according to Equation E3-2 or E3-3, using the torsional or flexural-torsional elastic buckling *stress*, F_e, determined as follows:

(i) For doubly symmetric members:

$$F_e = \left[\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right]\frac{1}{I_x + I_y} \tag{E4-4}$$

(ii) For singly symmetric members where y is the axis of symmetry:

$$F_e = \left(\frac{F_{ey} + F_{ez}}{2H}\right)\left[1 - \sqrt{1 - \frac{4F_{ey}F_{ez}H}{\left(F_{ey} + F_{ez}\right)^2}}\right] \tag{E4-5}$$

(iii) For unsymmetric members, F_e is the lowest root of the cubic equation:

$$(F_e - F_{ex})(F_e - F_{ey})(F_e - F_{ez}) - F_e^2(F_e - F_{ey})\left(\frac{x_o}{\bar{r}_o}\right)^2 - F_e^2(F_e - F_{ex})\left(\frac{y_o}{\bar{r}_o}\right)^2 = 0 \tag{E4-6}$$

where

A_g = gross area of member, in.^{2} (mm^2)

C_w = warping constant, in.^{6} (mm^6)

$$\bar{r}_o^2 = x_o^2 + y_o^2 + \frac{I_x + I_y}{A_g} \tag{E4-7}$$

$$H = 1 - \frac{x_o^2 + y_o^2}{\bar{r}_o^2} \tag{E4-8}$$

$$F_{ex} = \frac{\pi^2 E}{\left(\dfrac{K_x L}{r_x}\right)^2} \tag{E4-9}$$

$$F_{ey} = \frac{\pi^2 E}{\left(\dfrac{K_y L}{r_y}\right)^2} \tag{E4-10}$$

$$F_{ez} = \left(\frac{\pi^2 E C_w}{(K_z L)^2} + GJ\right)\frac{1}{A_g \bar{r}_o^2} \tag{E4-11}$$

G = shear modulus of elasticity of steel = 11,200 ksi (77 200 MPa)

I_x, I_y = moment of inertia about the principal axes, in.^{4} (mm^4)

J = torsional constant, in.^{4} (mm^4)

K_z = *effective length factor* for torsional buckling

x_o, y_o = coordinates of shear center with respect to the centroid, in. (mm)

$\bar{r}_o$ = polar radius of gyration about the shear center, in. (mm)

r_y = radius of gyration about y-axis, in. (mm)

User Note: For doubly symmetric I-shaped sections, C_w may be taken as $I_y h_o^2/4$, where h_o is the distance between flange centroids, in lieu of a more precise analysis. For tees and double angles, omit term with C_w when computing F_{ez} and take x_o as 0.

E5. SINGLE ANGLE COMPRESSION MEMBERS

The *nominal compressive strength*, P_n, of single angle members shall be determined in accordance with Section E3 or Section E7, as appropriate, for axially loaded members, as well as those subject to the slenderness modification of Section E5(a) or E5(b), provided the members meet the criteria imposed.

The effects of eccentricity on single angle members are permitted to be neglected when the members are evaluated as axially loaded compression members using one of the effective slenderness ratios specified below, provided that: (1) members are loaded at the ends in compression through the same one leg; (2) members are attached by welding or by minimum two-bolt *connections*; and (3) there are no intermediate transverse *loads.*

(a) For equal-leg angles or unequal-leg angles connected through the longer leg that are individual members or are web members of planar trusses with adjacent web members attached to the same side of the *gusset plate* or chord:

(i) When $0 \le \frac{L}{r_x} \le 80$:

$$\frac{KL}{r} = 72 + 0.75\frac{L}{r_x} \tag{E5-1}$$

(ii) When $\frac{L}{r_x} > 80$:

$$\frac{KL}{r} = 32 + 1.25\frac{L}{r_x} \le 200 \tag{E5-2}$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Equations E5-1 and E5-2 shall be increased by adding $4[(b_l/b_s)^2 - 1]$, but KL/r of the members shall not be less than $0.95L/r_z$.

(b) For equal-leg angles or unequal-leg angles connected through the longer leg that are web members of box or space trusses with adjacent web members attached to the same side of the *gusset plate* or chord:

(i) When $0 \le \frac{L}{r_x} \le 75$:

$$\frac{KL}{r} = 60 + 0.8\frac{L}{r_x} \tag{E5-3}$$

(ii) When $\frac{L}{r_x} > 75$:

$$\frac{KL}{r} = 45 + \frac{L}{r_x} \le 200 \tag{E5-4}$$

For unequal-leg angles with leg length ratios less than 1.7 and connected through the shorter leg, KL/r from Equations E5-3 and E5-4 shall be increased by adding $6[(b_l/b_s)^2 - 1]$, but KL/r of the member shall not be less than $0.82L/r_z$,

where

L = length of member between work points at truss chord centerlines, in. (mm)
b_l = longer leg of angle, in. (mm)
b_s = shorter leg of angle, in. (mm)
r_x = radius of gyration about *geometric axis* parallel to connected leg, in. (mm)
r_z = radius of gyration for the minor principal axis, in. (mm)

(c) Single angle members with different end conditions from those described in Section E5(a) or (b), with leg length ratios greater than 1.7, or with transverse loading shall be evaluated for combined axial *load* and flexure using the provisions of Chapter H. End connection to different legs on each end or to both legs, the use of single bolts or the attachment of adjacent web members to opposite sides of the *gusset plate* or chord shall constitute different end conditions requiring the use of Chapter H provisions.

E6. BUILT-UP MEMBERS

1. Compressive Strength

(a) The *nominal compressive strength* of *built-up members* composed of two or more shapes that are interconnected by bolts or welds shall be determined in accordance with Sections E3, E4, or E7 subject to the following modification. In lieu of more accurate analysis, if the buckling mode involves relative deformations that produce shear *forces* in the connectors between individual shapes, KL/r is replaced by $(KL/r)_m$ determined as follows:

(i) For intermediate connectors that are snug-tight bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + \left(\frac{a}{r_i}\right)^2} \qquad \text{(E6-1)}$$

(ii) For intermediate connectors that are welded or pretensioned bolted:

$$\left(\frac{KL}{r}\right)_m = \sqrt{\left(\frac{KL}{r}\right)_o^2 + 0.82\frac{\alpha^2}{(1+\alpha^2)}\left(\frac{a}{r_{ib}}\right)^2} \qquad \text{(E6-2)}$$

where

$\left(\frac{KL}{r}\right)_m$ = modified *column* slenderness of *built-up member*

$\left(\frac{KL}{r}\right)_o$ = column slenderness of built-up member acting as a unit in the buckling direction being considered

a = distance between connectors, in. (mm)

r_i = minimum radius of gyration of individual component, in. (mm)

r_{ib} = radius of gyration of individual component relative to its centroidal axis parallel to member axis of buckling, in. (mm)

α = separation ratio = $h/2r_{ib}$

h = distance between centroids of individual components perpendicular to the member axis of buckling, in. (mm)

(b) The nominal compressive strength of built-up members composed of two or more shapes or plates with at least one open side interconnected by perforated *cover plates* or *lacing* with *tie plates* shall be determined in accordance with Sections E3, E4, or E7 subject to the modification given in Section E6.1(a).

2. Dimensional Requirements

Individual components of compression members composed of two or more shapes shall be connected to one another at intervals, a, such that the effective slenderness ratio Ka/r_i of each of the component shapes, between the *fasteners*, does not exceed three-fourths times the governing slenderness ratio of the *built-up member*. The least radius of gyration, r_i, shall be used in computing the slenderness ratio of each component part. The end *connection* shall be welded or pretensioned bolted with Class A or B *faying surfaces*.

> **User Note:** It is acceptable to design a bolted end *connection* of a built-up compression member for the full compressive *load* with bolts in shear and bolt values based on bearing values; however, the bolts must be pretensioned. The requirement for Class A or B faying surfaces is not intended for the resistance of the axial force in the built-up member, but rather to prevent relative movement between the components at the end as the built-up member takes a curved shape.

At the ends of built-up compression members bearing on base plates or *milled surfaces*, all components in contact with one another shall be connected by a weld having a length not less than the maximum width of the member or by bolts spaced longitudinally not more than four diameters apart for a distance equal to $1^1/_2$ times the maximum width of the member.

Along the length of built-up compression members between the end connections required above, longitudinal spacing for intermittent welds or bolts shall be adequate to provide for the transfer of the required *force*s. For limitations on the longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates, see Section J3.5. Where a component of a built-up compression member consists of an outside plate, the maximum spacing shall not exceed the thickness of the thinner outside plate times $0.75\sqrt{E/F_y}$, nor 12 in. (305 mm), when intermittent welds are provided along the edges of the components or when fasteners are provided on all *gage* lines at each section. When fasteners are staggered, the maximum spacing on each gage line shall not exceed the thickness of the thinner outside plate times $1.12\sqrt{E/F_y}$ nor 18 in. (460 mm).

Open sides of compression members built up from plates or shapes shall be provided with continuous *cover plates* perforated with a succession of access holes. The unsupported width of such plates at access holes, as defined in Section B4, is assumed to contribute to the *available strength* provided the following requirements are met:

(1) The width-thickness ratio shall conform to the limitations of Section B4.

> **User Note:** It is conservative to use the limiting width/thickness ratio for Case 14 in Table B4.1 with the width, b, taken as the transverse distance between the nearest lines of fasteners. The net area of the plate is taken at the widest hole. In lieu of this approach, the limiting width thickness ratio may be determined through analysis.

(2) The ratio of length (in direction of *stress*) to width of hole shall not exceed two.
(3) The clear distance between holes in the direction of stress shall be not less than the transverse distance between nearest lines of connecting fasteners or welds.
(4) The periphery of the holes at all points shall have a minimum radius of $1^1/_2$ in. (38 mm).

As an alternative to perforated cover plates, *lacing* with *tie plate*s is permitted at each end and at intermediate points if the lacing is interrupted. Tie plates shall be as near the ends as practicable. In members providing *available strength*, the end tie plates shall have a length of not less than the distance between the lines of fasteners or welds connecting them to the components of the member. Intermediate tie plates shall have a length not less than one-half of this distance. The thickness of tie plates shall be not less than one-fiftieth of the distance between lines of welds or fasteners connecting them to the segments of the members. In welded construction, the welding on each line connecting a tie plate shall total not less than one-third the length of the plate. In bolted construction, the spacing in the direction of stress in tie plates shall be not more than six diameters and the tie plates shall be connected to each segment by at least three fasteners.

Lacing, including flat bars, angles, channels, or other shapes employed as lacing, shall be so spaced that the L/r ratio of the flange included between their connections shall not exceed three-fourths times the governing slenderness ratio for the member as a whole. Lacing shall be proportioned to provide a shearing strength normal to the axis of the member equal to 2 percent of the *available compressive strength* of the member. The L/r ratio for lacing bars arranged in single systems shall not exceed 140. For double lacing this ratio shall not exceed 200. Double lacing bars shall be joined at the intersections. For lacing bars in compression, l is permitted to be taken as the unsupported length of the *lacing* bar between welds or fasteners connecting it to the components of the *built-up member* for single lacing, and 70 percent of that distance for double lacing.

User Note: The inclination of lacing bars to the axis of the member shall preferably be not less than 60° for single lacing and 45° for double lacing. When the distance between the lines of welds or fasteners in the flanges is more than 15 in. (380 mm), the lacing shall preferably be double or be made of angles.

For additional spacing requirements, see Section J3.5.

E7. MEMBERS WITH SLENDER ELEMENTS

This section applies to compression members with slender sections, as defined in Section B4 for uniformly compressed elements.

The *nominal compressive strength*, P_n, shall be determined based on the *limit states* of *flexural*, *torsional* and *flexural-torsional buckling*.

$$P_n = F_{cr} A_g \tag{E7-1}$$

(a) When $\dfrac{KL}{r} \leq 4.71\sqrt{\dfrac{E}{QF_y}}$ (or $F_e \geq 0.44QF_y$)

$$F_{cr} = Q\left[0.658^{\frac{QF_y}{F_e}}\right]F_y \tag{E7-2}$$

(b) When $\dfrac{KL}{r} > 4.71\sqrt{\dfrac{E}{QF_y}}$ (or $F_e < 0.44QF_y$)

$$F_{cr} = 0.877F_e \tag{E7-3}$$

where

F_e = elastic critical buckling stress, calculated using Equations E3-4 and E4-4 for doubly symmetric members, Equations E3-4 and E4-5 for singly symmetric members, and Equation E4-6 for unsymmetric members, except for single angles where F_e is calculated using Equation E3-4.

Q = 1.0 for members with *compact* and *noncompact sections*, as defined in Section B4, for uniformly compressed elements

= Q_sQ_a for members with *slender-element section*s, as defined in Section B4, for uniformly compressed elements.

User Note: For cross sections composed of only unstiffened slender elements, $Q = Q_s$ ($Q_a = 1.0$). For cross sections composed of only stiffened slender elements, $Q = Q_a$ ($Q_s = 1.0$). For cross sections composed of both stiffened and unstiffened slender elements, $Q = Q_sQ_a$.

1. Slender Unstiffened Elements, Q_s

The reduction factor Q_s for slender *unstiffened elements* is defined as follows:

(a) For flanges, angles, and plates projecting from rolled *column*s or other compression members:

(i) When $\dfrac{b}{t} \leq 0.56\sqrt{\dfrac{E}{F_y}}$

$$Q_s = 1.0 \tag{E7-4}$$

(ii) When $0.56\sqrt{E/F_y} < b/t < 1.03\sqrt{E/F_y}$

$$Q_s = 1.415 - 0.74\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \tag{E7-5}$$

(iii) When $b/t \geq 1.03\sqrt{E/F_y}$

$$Q_s = \frac{0.69E}{F_y\left(\dfrac{b}{t}\right)^2} \tag{E7-6}$$

(b) For flanges, angles, and plates projecting from built-up columns or other compression members:

(i) When $\frac{b}{t} \leq 0.64\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.0 \tag{E7-7}$$

(ii) When $0.64\sqrt{\frac{Ek_c}{F_y}} < b/t \leq 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = 1.415 - 0.65\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{Ek_c}} \tag{E7-8}$$

(iii) When $b/t > 1.17\sqrt{\frac{Ek_c}{F_y}}$

$$Q_s = \frac{0.90Ek_c}{F_y\left(\frac{b}{t}\right)^2} \tag{E7-9}$$

where

$k_c = \frac{4}{\sqrt{h/t_w}}$, and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

(c) For single angles

(i) When $\frac{b}{t} \leq 0.45\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \tag{E7-10}$$

(ii) When $0.45\sqrt{E/F_y} < b/t \leq 0.91\sqrt{E/F_y}$

$$Q_s = 1.34 - 0.76\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}} \tag{E7-11}$$

(iii) When $b/t > 0.91\sqrt{E/F_y}$

$$Q_s = \frac{0.53E}{F_y\left(\frac{b}{t}\right)^2} \tag{E7-12}$$

where

b = full width of longest angle leg, in. (mm)

(d) For stems of tees

(i) When $\frac{d}{t} \leq 0.75\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.0 \tag{E7-13}$$

(ii) When $0.75\sqrt{\frac{E}{F_y}} < d/t \leq 1.03\sqrt{\frac{E}{F_y}}$

$$Q_s = 1.908 - 1.22\left(\frac{d}{t}\right)\sqrt{\frac{F_y}{E}} \tag{E7-14}$$

(iii) When $d/t > 1.03\sqrt{\frac{E}{F_y}}$

$$Q_s = \frac{0.69E}{F_y\left(\frac{d}{t}\right)^2} \tag{E7-15}$$

where

b = width of unstiffened compression element, as defined in Section B4, in. (mm)
d = the full nominal depth of tee, in. (mm)
t = thickness of element, in. (mm)

2. Slender Stiffened Elements, Q_a

The reduction factor, Q_a, for slender *stiffened elements* is defined as follows:

$$Q_a = \frac{A_{eff}}{A} \tag{E7-16}$$

where

A = total cross-sectional area of member, in.^{2} (mm^2)
A_{eff} = summation of the effective areas of the cross section based on the reduced *effective width*, b_e, in.^{2} (mm^2)

The reduced effective width, b_e, is determined as follows:

(a) For uniformly compressed slender elements, with $\frac{b}{t} \geq 1.49\sqrt{\frac{E}{f}}$, except flanges of square and rectangular sections of uniform thickness:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.34}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \tag{E7-17}$$

where

f is taken as F_{cr} with F_{cr} calculated based on $Q = 1.0$.

(b) For flanges of square and rectangular *slender-element sections* of uniform thickness with $\frac{b}{t} \geq 1.40\sqrt{\frac{E}{f}}$:

$$b_e = 1.92t\sqrt{\frac{E}{f}}\left[1 - \frac{0.38}{(b/t)}\sqrt{\frac{E}{f}}\right] \leq b \tag{E7-18}$$

where

$f = P_n / A_{eff}$

User Note: In lieu of calculating $f = P_n/A_{eff}$, which requires iteration, f may be taken equal to F_y. This will result in a slightly conservative estimate of column capacity.

(c) For axially-loaded circular sections:

When $0.11\frac{E}{F_y} < \frac{D}{t} < 0.45\frac{E}{F_y}$

$$Q = Q_a = \frac{0.038E}{F_y(D/t)} + \frac{2}{3} \tag{E7-19}$$

where

D = outside diameter, in. (mm)
t = wall thickness, in. (mm)

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

This chapter applies to members subject to simple bending about one principal axis. For simple bending, the member is loaded in a plane parallel to a principal axis that passes through the shear center or is restrained against twisting at *load* points and supports.

The chapter is organized as follows:

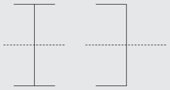
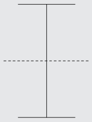
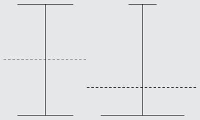
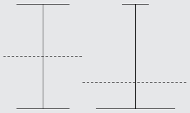
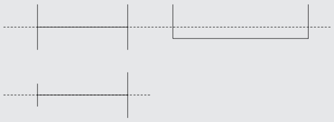
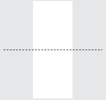
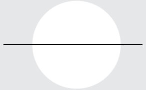
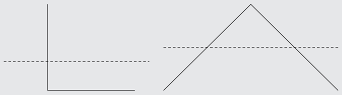
F1. General Provisions
F2. Doubly Symmetric Compact I-Shaped Members and Channels Bent about Their Major Axis
F3. Doubly Symmetric I-Shaped Members with Compact Webs and Noncompact or Slender Flanges Bent about Their Major Axis
F4. Other I-Shaped Members with Compact or Noncompact Webs Bent about Their Major Axis
F5. Doubly Symmetric and Singly Symmetric I-Shaped Members with Slender Webs Bent about Their Major Axis
F6. I-Shaped Members and Channels Bent about Their Minor Axis
F7. Square and Rectangular HSS and Box-Shaped Members
F8. Round HSS
F9. Tees and Double Angles Loaded in the Plane of Symmetry
F10. Single Angles
F11. Rectangular Bars and Rounds
F12. Unsymmetrical Shapes
F13. Proportions of Beams and Girders

User Note: For members not included in this chapter the following sections apply:

• H1–H3.	Members subject to biaxial flexure or to combined flexure and axial force.
• H4.	Members subject to flexure and torsion.
• Appendix 3.	Members subject to fatigue.
• Chapter G.	Design provisions for shear.

For guidance in determining the appropriate sections of this chapter to apply, Table User Note F1.1 may be used.

TABLE User Note F1.1
Selection Table for the Application of Chapter F Sections

Section in Chapter F	Cross Section	Flange Slenderness	Web Slenderness	Limit States
F2		C	C	Y, LTB
F3		NC, S	C	LTB, FLB
F4		C, NC, S	C, NC	Y, LTB, FLB, TFY
F5		C, NC, S	S	Y, LTB, FLB, TFY
F6		C, NC, S	N/A	Y, FLB
F7		C, NC, S	C, NC	Y, FLB, WLB
F8		N/A	N/A	Y, LB
F9		C, NC, S	N/A	Y, LTB, FLB
F10		N/A	N/A	Y, LTB, LLB
F11		N/A	N/A	Y, LTB
F12	Unsymmetrical shapes	N/A	N/A	All limit states

Y = yielding, LTB = lateral-torsional buckling, FLB = flange local buckling, WLB = web local buckling, TFY = tension flange yielding, LLB = leg local buckling, LB = local buckling, C = compact, NC = noncompact, S = slender

F1. GENERAL PROVISIONS

The *design flexural strength*, $\phi_b M_n$, and the *allowable flexural strength*, M_n/Ω_b, shall be determined as follows:

(1) For all provisions in this chapter

$$\phi_b = 0.90\,(\text{LRFD}) \qquad \Omega_b = 1.67\,(\text{ASD})$$

and the *nominal flexural strength*, M_n, shall be determined according to Sections F2 through F12.

(2) The provisions in this chapter are based on the assumption that points of support for *beams* and girders are restrained against rotation about their longitudinal axis.

The following terms are common to the equations in this chapter except where noted:

C_b = *lateral-torsional buckling* modification factor for nonuniform moment diagrams when both ends of the unsupported segment are braced

$$C_b = \frac{12.5M_{\max}}{2.5M_{\max} + 3M_A + 4M_B + 3M_C} R_m \leq 3.0 \qquad \text{(F1-1)}$$

where

$M_{\max}$ = absolute value of maximum moment in the unbraced segment, kip-in. (N-mm)

M_A = absolute value of moment at quarter point of the unbraced segment, kip-in. (N-mm)

M_B = absolute value of moment at centerline of the unbraced segment, kip-in. (N-mm)

M_C = absolute value of moment at three-quarter point of the unbraced segment, kip-in. (N-mm)

R_m = cross-section monosymmetry parameter
= 1.0, doubly symmetric members
= 1.0, singly symmetric members subjected to *single curvature* bending
= $0.5 + 2\left(\frac{I_{yc}}{I_y}\right)^2$, singly symmetric members subjected to *reverse curvature* bending

I_y = moment of inertia about the principal y-axis, in.^{4} (mm^4)

I_{yc} = moment of inertia about y-axis referred to the compression flange, or if reverse curvature bending, referred to the smaller flange, in.^{4} (mm^4)

In singly symmetric members subjected to *reverse curvature* bending, the *lateral-torsional buckling* strength shall be checked for both flanges. The available flexural strength shall be greater than or equal to the maximum required moment causing compression within the flange under consideration.

C_b is permitted to be conservatively taken as 1.0 for all cases. For cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$.

User Note: For doubly symmetric members with no transverse loading between brace points, Equation F1-1 reduces to 2.27 for the case of equal end moments of opposite sign and to 1.67 when one end moment equals zero.

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section B4.

User Note: All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5, and M4×6 have compact flanges for $F_y \leq 50$ ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \leq 65$ ksi (450 MPa).

The *nominal flexural strength*, M_n, shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and *lateral-torsional buckling*.

1. Yielding

$$M_n = M_p = F_y Z_x \tag{F2-1}$$

where

F_y = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
Z_x = plastic section modulus about the x-axis, in.^{3} (mm^3)

2. Lateral-Torsional Buckling

(a) When $L_b \leq L_p$, the limit state of lateral-torsional buckling does not apply.
(b) When $L_p < L_b \leq L_r$

$$M_n = C_b \left[M_p - (M_p - 0.7F_y S_x) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq M_p \tag{F2-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_x \leq M_p \tag{F2-3}$$

where

L_b = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm)

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \tag{F2-4}$$

and where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
J = torsional constant, in.^{4} (mm^4)
S_x = elastic section modulus taken about the x-axis, in.^{3} (mm^3)

User Note: The square root term in Equation F2-4 may be conservatively taken equal to 1.0.

The limiting lengths L_p and L_r are determined as follows:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \tag{F2-5}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{0.7F_y}{E} \frac{S_x h_o}{Jc} \right)^2}} \tag{F2-6}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \tag{F2-7}$$

and

For a doubly symmetric I-shape: $c = 1$ (F2-8a)

For a channel: $c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}}$ (F2-8b)

where

h_o = distance between the flange centroids, in. (mm)

User Note: If the square root term in Equation F2-4 is conservatively taken equal to 1, Equation F2-6 becomes

$$L_r = \pi r_{ts} \sqrt{\frac{E}{0.7F_y}}$$

Note that this approximation can be extremely conservative.

For doubly symmetric I-shapes with rectangular flanges, $C_w = \frac{I_y h_o^2}{4}$ and thus Equation F2-7 becomes

$$r_{ts}^2 = \frac{I_y h_o}{2S_x}$$

r_{ts} may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-sixth of the web:

$$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \frac{1}{6}\frac{h t_w}{b_f t_f}\right)}}$$

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members bent about their major axis having compact webs and noncompact or slender flanges as defined in Section B4.

> **User Note:** The following shapes have noncompact flanges for F_y = 50 ksi (345 MPa): W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6 × 8.5, and M4×6. All other ASTM A6 W, S, M, and HP shapes have compact flanges for $F_y \le 50$ ksi (345 MPa).

The nominal flexural strength, M_n, shall be the lower value obtained according to the *limit states* of *lateral-torsional buckling* and compression flange *local buckling*.

1. Lateral-Torsional Buckling

For *lateral-torsional buckling*, the provisions of Section F2.2 shall apply.

2. Compression Flange Local Buckling

(a) For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7F_yS_x)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \qquad \text{(F3-1)}$$

(b) For sections with slender flanges

$$M_n = \frac{0.9Ek_cS_x}{\lambda^2} \qquad \text{(F3-2)}$$

where

$\lambda = \dfrac{b_f}{2t_f}$

$\lambda_{pf} = \lambda_p$ is the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$ is the limiting slenderness for a noncompact flange, Table B4.1

$k_c = \dfrac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to: (a) doubly symmetric I-shaped members bent about their major axis with noncompact webs; and (b) singly symmetric I-shaped members with webs attached to the mid-width of the flanges, bent about their major axis, with compact or noncompact webs, as defined in Section B4.

> **User Note:** I-shaped members for which this section is applicable may be designed conservatively using Section F5.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of compression flange *yielding*, *lateral-torsional buckling*, compression flange *local buckling* and tension flange yielding.

1. Compression Flange Yielding

$$M_n = R_{pc} M_{yc} = R_{pc} F_y S_{xc} \tag{F4-1}$$

2. Lateral-Torsional Buckling

(a) When $L_b \leq L_p$, the *limit state* of *lateral-torsional buckling* does not apply.
(b) When $L_p < L_b \leq L_r$

$$M_n = C_b \left[R_{pc} M_{yc} - \left(R_{pc} M_{yc} - F_L S_{xc} \right) \left(\frac{L_b - L_p}{L_r - L_p} \right) \right] \leq R_{pc} M_{yc} \tag{F4-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_{xc} \leq R_{pc} M_{yc} \tag{F4-3}$$

where

$$M_{yc} = F_y S_{xc} \tag{F4-4}$$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_t} \right)^2} \sqrt{1 + 0.078 \frac{J}{S_{xc} h_o} \left(\frac{L_b}{r_t} \right)^2} \tag{F4-5}$$

For $\frac{I_{yc}}{I_y} \leq 0.23$, J shall be taken as zero.

The stress, F_L, is determined as follows:

(i) For $\frac{S_{xt}}{S_{xc}} \geq 0.7$

$$F_L = 0.7 F_y \tag{F4-6a}$$

(ii) For $\frac{S_{xt}}{S_{xc}} < 0.7$

$$F_L = F_y \frac{S_{xt}}{S_{xc}} \geq 0.5 F_y \tag{F4-6b}$$

The limiting laterally unbraced length for the limit state of *yielding*, L_p, is

$$L_p = 1.1 r_t \sqrt{\frac{E}{F_y}} \tag{F4-7}$$

The limiting unbraced length for the limit state of inelastic lateral-torsional buckling, L_r, is

$$L_r = 1.95 r_t \frac{E}{F_L} \sqrt{\frac{J}{S_{xc} h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left(\frac{F_L}{E} \frac{S_{xc} h_o}{J} \right)^2}} \tag{F4-8}$$

The web *plastification* factor, R_{pc}, is determined as follows:

(i) For $\dfrac{h_c}{t_w} \le \lambda_{pw}$

$$R_{pc} = \frac{M_p}{M_{yc}} \tag{F4-9a}$$

(ii) For $\dfrac{h_c}{t_w} > \lambda_{pw}$

$$R_{pc} = \left[\frac{M_p}{M_{yc}} - \left(\frac{M_p}{M_{yc}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yc}} \tag{F4-9b}$$

where

M_p $= Z_x F_y \le 1.6 S_{xc} F_y$
S_{xc}, S_{xt} = elastic section modulus referred to tension and compression flanges, respectively, in.^{3} (mm^3)
λ $= \dfrac{h_c}{t_w}$
λ_{pw} $= \lambda_p$, the limiting slenderness for a compact web, Table B4.1
λ_{rw} $= \lambda_r$, the limiting slenderness for a noncompact web, Table B4.1

The effective radius of gyration for lateral-torsional buckling, r_t, is determined as follows:

(i) For I-shapes with a rectangular compression flange:

$$r_t = \frac{b_{fc}}{\sqrt{12\left(\dfrac{h_o}{d} + \dfrac{1}{6}a_w\dfrac{h^2}{h_o d}\right)}} \tag{F4-10}$$

where

$$a_w = \frac{h_c t_w}{b_{fc} t_{fc}} \tag{F4-11}$$

b_{fc} = compression flange width, in. (mm)
t_{fc} = compression flange thickness, in. (mm)

(ii) For I-shapes with channel caps or *cover plates* attached to the compression flange:

r_t = radius of gyration of the flange components in flexural compression plus one-third of the web area in compression due to application of major axis bending moment alone, in. (mm)
a_w = the ratio of two times the web area in compression due to application of major axis bending moment alone to the area of the compression flange components

User Note: For I-shapes with a rectangular compression flange, r_t may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-third of the compression portion of the web; in other words,

$$r_t = \frac{b_{fc}}{\sqrt{12\left(1 + \frac{1}{6}a_w\right)}}$$

3. Compression Flange Local Buckling

(a) For sections with compact flanges, the *limit state* of *local buckling* does not apply.

(b) For sections with noncompact flanges

$$M_n = \left[R_{pc}M_{yc} - (R_{pc}M_{yc} - F_L S_{xc})\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \quad \text{(F4-12)}$$

(c) For sections with slender flanges

$$M_n = \frac{0.9Ek_c S_{xc}}{\lambda^2} \quad \text{(F4-13)}$$

where

F_L is defined in Equations F4-6a and F4-6b

R_{pc} is the web *plastification* factor, determined by Equations F4-9

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \frac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table B4.1

4. Tension Flange Yielding

(a) When $S_{xt} \geq S_{xc}$, the *limit state* of tension flange *yielding* does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = R_{pt}M_{yt} \quad \text{(F4-14)}$$

where

$$M_{yt} = F_y S_{xt}$$

The web *plastification* factor corresponding to the tension flange yielding limit state, R_{pt}, is determined as follows:

(i) For $\frac{h_c}{t_w} \leq \lambda_{pw}$

$$R_{pt} = \frac{M_p}{M_{yt}} \quad \text{(F4-15a)}$$

(ii) For $\dfrac{h_c}{t_w} > \lambda_{pw}$

$$R_{pt} = \left[\frac{M_p}{M_{yt}} - \left(\frac{M_p}{M_{yt}} - 1\right)\left(\frac{\lambda - \lambda_{pw}}{\lambda_{rw} - \lambda_{pw}}\right)\right] \le \frac{M_p}{M_{yt}} \qquad \text{(F4-15b)}$$

where

$\lambda = \dfrac{h_c}{t_w}$

$\lambda_{pw} = \lambda_p$, the limiting slenderness for a compact web, defined in Table B4.1

$\lambda_{rw} = \lambda_r$, the limiting slenderness for a noncompact web, defined in Table B4.1

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric and singly symmetric I-shaped members with slender webs attached to the mid-width of the flanges, bent about their major axis, as defined in Section B4.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of compression flange *yielding*, *lateral-torsional buckling*, compression flange *local buckling* and tension flange yielding.

1. Compression Flange Yielding

$$M_n = R_{pg} F_y S_{xc} \qquad \text{(F5-1)}$$

2. Lateral-Torsional Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \qquad \text{(F5-2)}$$

(a) When $L_b \le L_p$, the *limit state* of *lateral-torsional buckling* does not apply.

(b) When $L_p < L_b \le L_r$

$$F_{cr} = C_b \left[F_y - (0.3F_y)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le F_y \qquad \text{(F5-3)}$$

(c) When $L_b > L_r$

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\dfrac{L_b}{r_t}\right)^2} \le F_y \qquad \text{(F5-4)}$$

where

L_p is defined by Equation F4-7

$$L_r = \pi r_t \sqrt{\frac{E}{0.7F_y}} \qquad \text{(F5-5)}$$

R_{pg} is the bending strength reduction factor:

$$R_{pg} = 1 - \frac{a_w}{1200 + 300a_w}\left(\frac{h_c}{t_w} - 5.7\sqrt{\frac{E}{F_y}}\right) \le 1.0 \qquad \text{(F5-6)}$$

a_w is defined by Equation F4-11 but shall not exceed 10

and

r_t is the effective radius of gyration for lateral buckling as defined in Section F4.

3. Compression Flange Local Buckling

$$M_n = R_{pg} F_{cr} S_{xc} \qquad \text{(F5-7)}$$

(a) For sections with compact flanges, the *limit state* of compression flange *local buckling* does not apply.

(b) For sections with noncompact flanges

$$F_{cr} = \left[F_y - (0.3F_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \qquad \text{(F5-8)}$$

(c) For sections with slender flange sections

$$F_{cr} = \frac{0.9Ek_c}{\left(\frac{b_f}{2t_f}\right)^2} \qquad \text{(F5-9)}$$

where

$k_c = \frac{4}{\sqrt{h/t_w}}$ and shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes

$\lambda = \frac{b_{fc}}{2t_{fc}}$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table B4.1

4. Tension Flange Yielding

(a) When $S_{xt} \ge S_{xc}$, the *limit state* of tension flange *yielding* does not apply.

(b) When $S_{xt} < S_{xc}$

$$M_n = F_y S_{xt} \qquad \text{(F5-10)}$$

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

This section applies to I-shaped members and channels bent about their minor axis.

The nominal flexural strength, M_n, shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and flange *local buckling*.

1. Yielding

$$M_n = M_p = F_y Z_y \le 1.6F_y S_y \qquad \text{(F6-1)}$$

2. Flange Local Buckling

(a) For sections with compact flanges the limit state of yielding shall apply.

> **User Note:** All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5, and M4×6 have compact flanges at $F_y \leq 50$ ksi (345 MPa).

(b) For sections with noncompact flanges

$$M_n = \left[M_p - (M_p - 0.7F_yS_y)\left(\frac{\lambda - \lambda_{pf}}{\lambda_{rf} - \lambda_{pf}}\right)\right] \tag{F6-2}$$

(c) For sections with slender flanges

$$M_n = F_{cr}S_y \tag{F6-3}$$

where

$$F_{cr} = \frac{0.69E}{\left(\dfrac{b_f}{2t_f}\right)^2} \tag{F6-4}$$

$\lambda = \dfrac{b}{t}$

$\lambda_{pf} = \lambda_p$, the limiting slenderness for a compact flange, Table B4.1

$\lambda_{rf} = \lambda_r$, the limiting slenderness for a noncompact flange, Table B4.1

S_y for a channel shall be taken as the minimum section modulus

F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

This section applies to square and rectangular *HSS*, and doubly symmetric box-shaped members bent about either axis, having compact or noncompact webs and compact, noncompact or slender flanges as defined in Section B4.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, flange *local buckling* and web *local buckling* under pure flexure.

1. Yielding

$$M_n = M_p = F_yZ \tag{F7-1}$$

where

Z = plastic section modulus about the axis of bending, in.^{3} (mm^3)

2. Flange Local Buckling

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.

(b) For sections with noncompact flanges

$$M_n = M_p - (M_p - F_yS)\left(3.57\frac{b}{t}\sqrt{\frac{F_y}{E}} - 4.0\right) \leq M_p \tag{F7-2}$$

(c) For sections with slender flanges

$$M_n = F_y S_{eff} \tag{F7-3}$$

where

S_{eff} is the *effective section modulus* determined with the *effective width* of the compression flange taken as:

$$b_e = 1.92t\sqrt{\frac{E}{F_y}}\left[1 - \frac{0.38}{b/t}\sqrt{\frac{E}{F_y}}\right] \leq b \tag{F7-4}$$

3. Web Local Buckling

(a) For *compact sections*, the *limit state* of web *local buckling* does not apply.
(b) For sections with noncompact webs

$$M_n = M_p - (M_p - F_y S_x)\left(0.305\frac{h}{t_w}\sqrt{\frac{F_y}{E}} - 0.738\right) \leq M_p \tag{F7-5}$$

F8. ROUND HSS

This section applies to round *HSS* having D/t ratios of less than $\dfrac{0.45E}{F_y}$.

The nominal flexural strength, M_n, shall be the lower value obtained according to the *limit states* of *yielding (plastic moment)* and *local buckling*.

1. Yielding

$$M_n = M_p = F_y Z \tag{F8-1}$$

2. Local Buckling

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.
(b) For noncompact sections

$$M_n = \left(\frac{0.021E}{\dfrac{D}{t}} + F_y\right) S \tag{F8-2}$$

(c) For sections with slender walls

$$M_n = F_{cr} S \tag{F8-3}$$

where

$$F_{cr} = \frac{0.33E}{\dfrac{D}{t}} \tag{F8-4}$$

S = elastic section modulus, in.^{3} (mm^3)

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

This section applies to tees and double angles loaded in the plane of symmetry.

The nominal flexural strength, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, *lateral-torsional buckling* and flange *local buckling*.

1. Yielding

$$M_n = M_p \tag{F9-1}$$

where

$$M_p = F_y Z_x \leq 1.6 M_y \quad \text{for stems in tension} \tag{F9-2}$$

$$\leq M_y \quad \text{for stems in compression} \tag{F9-3}$$

2. Lateral-Torsional Buckling

$$M_n = M_{cr} = \frac{\pi\sqrt{EI_y GJ}}{L_b}\left[B + \sqrt{1 + B^2}\right] \tag{F9-4}$$

where

$$B = \pm\, 2.3\left(\frac{d}{L_b}\right)\sqrt{\frac{I_y}{J}} \tag{F9-5}$$

The plus sign for B applies when the stem is in tension and the minus sign applies when the stem is in compression. If the tip of the stem is in compression anywhere along the *unbraced length*, the negative value of B shall be used.

3. Flange Local Buckling of Tees

$$M_n = F_{cr} S_{xc} \tag{F9-6}$$

S_{xc} is the elastic section modulus referred to the compression flange.
F_{cr} is determined as follows:

(a) For *compact sections*, the *limit state* of flange *local buckling* does not apply.
(b) For noncompact sections

$$F_{cr} = F_y\left(1.19 - 0.50\left(\frac{b_f}{2t_f}\right)\sqrt{\frac{F_y}{E}}\right) \tag{F9-7}$$

(c) For slender sections

$$F_{cr} = \frac{0.69E}{\left(\frac{b_f}{2t_f}\right)^2} \tag{F9-8}$$

F10. SINGLE ANGLES

This section applies to single angles with and without continuous lateral restraint along their length.

Single angles with continuous lateral-torsional restraint along the length shall be permitted to be designed on the basis of *geometric axis* (x, y) bending. Single angles without continuous lateral-torsional restraint along the length shall be designed using the provisions for *principal axis* bending except where the provision for bending about a geometric axis is permitted.

User Note: For geometric axis design, use section properties computed about the x- and y-axis of the angle, parallel and perpendicular to the legs. For principal axis design use section properties computed about the major and minor principal axes of the angle.

The *nominal flexural strength*, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (plastic moment)*, *lateral-torsional buckling* and leg *local buckling*.

1. Yielding

$$M_n = 1.5M_y \tag{F10-1}$$

where

$M_y =$ *yield moment* about the axis of bending, kip-in. (N-mm)

2. Lateral-Torsional Buckling

For single angles without continuous lateral-torsional restraint along the length

(a) When $M_e \le M_y$

$$M_n = \left(0.92 - \frac{0.17M_e}{M_y}\right)M_e \tag{F10-2}$$

(b) When $M_e > M_y$

$$M_n = \left(1.92 - 1.17\sqrt{\frac{M_y}{M_e}}\right)M_y \le 1.5M_y \tag{F10-3}$$

where

M_e, the elastic *lateral-torsional buckling* moment, is determined as follows:

(i) For bending about one of the *geometric axes* of an equal-leg angle with no lateral-torsional restraint

(a) With maximum compression at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2}\left(\sqrt{1 + 0.78\left(\frac{Lt}{b^2}\right)^2} - 1\right) \tag{F10-4a}$$

(b) With maximum tension at the toe

$$M_e = \frac{0.66Eb^4tC_b}{L^2}\left(\sqrt{1 + 0.78\left(\frac{Lt}{b^2}\right)^2} + 1\right) \tag{F10-4b}$$

M_y shall be taken as 0.80 times the *yield moment* calculated using the geometric section modulus.

User Note: M_n may be taken as M_y for single angles with their vertical leg toe in compression, and having a span-to-depth ratio less than or equal to

$$\frac{1.64E}{F_y}\sqrt{\left(\frac{t}{b}\right)^2 - 1.4\frac{F_y}{E}}.$$

(ii) For bending about one of the geometric axes of an equal-leg angle with lateral-torsional restraint at the point of maximum moment only

M_e shall be taken as 1.25 times M_e computed using Equation F10-4a or F10-4b.

M_y shall be taken as the yield moment calculated using the geometric section modulus.

(iii) For bending about the major principal axis of equal-leg angles:

$$M_e = \frac{0.46Eb^2t^2C_b}{L} \tag{F10-5}$$

(iv) For bending about the major principal axis of unequal-leg angles:

$$M_e = \frac{4.9EI_zC_b}{L^2}\left(\sqrt{\beta_w^2 + 0.052\left(\frac{Lt}{r_z}\right)^2} + \beta_w\right) \tag{F10-6}$$

where

C_b is computed using Equation F1-1 with a maximum value of 1.5.
L = laterally *unbraced length* of a member, in. (mm)
I_z = minor principal axis moment of inertia, in.^{4} (mm^4)
r_z = radius of gyration for the minor principal axis, in. (mm)
t = angle leg thickness, in. (mm)
β_w = a section property for unequal leg angles, positive for short legs in compression and negative for long legs in compression. If the long leg is in compression anywhere along the unbraced length of the member, the negative value of β_w shall be used.

User Note: The equation for β_w and values for common angle sizes are listed in the Commentary.

3. Leg Local Buckling

The *limit state* of leg *local buckling* applies when the toe of the leg is in compression.

(a) For *compact sections,* the limit state of leg local buckling does not apply.
(b) For sections with noncompact legs

$$M_n = F_yS_c\left(2.43 - 1.72\left(\frac{b}{t}\right)\sqrt{\frac{F_y}{E}}\right) \tag{F10-7}$$

(c) For sections with slender legs

$$M_n = F_{cr} S_c \tag{F10-8}$$

where

$$F_{cr} = \frac{0.71E}{\left(\frac{b}{t}\right)^2} \tag{F10-9}$$

b = outside width of leg in compression, in. (mm)

S_c = elastic section modulus to the toe in compression relative to the axis of bending, in.^{3} (mm^3). For bending about one of the *geometric axes* of an equal-leg angle with no lateral-torsional restraint, S_c shall be 0.80 of the geometric axis section modulus.

F11. RECTANGULAR BARS AND ROUNDS

This section applies to rectangular bars bent about either *geometric axis* and rounds.

The *nominal flexural strength*, M_n, shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and *lateral-torsional buckling*, as required.

1. Yielding

For rectangular bars with $\frac{L_b d}{t^2} \leq \frac{0.08E}{F_y}$ bent about their major axis, rectangular bars bent about their minor axis, and rounds:

$$M_n = M_p = F_y Z \leq 1.6 M_y \tag{F11-1}$$

2. Lateral-Torsional Buckling

(a) For rectangular bars with $\frac{0.08E}{F_y} < \frac{L_b d}{t^2} \leq \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = C_b \left[1.52 - 0.274 \left(\frac{L_b d}{t^2} \right) \frac{F_y}{E} \right] M_y \leq M_p \tag{F11-2}$$

(b) For rectangular bars with $\frac{L_b d}{t^2} > \frac{1.9E}{F_y}$ bent about their major axis:

$$M_n = F_{cr} S_x \leq M_p \tag{F11-3}$$

where

$$F_{cr} = \frac{1.9 E C_b}{\frac{L_b d}{t^2}} \tag{F11-4}$$

t = width of rectangular bar parallel to axis of bending, in. (mm)

d = depth of rectangular bar, in. (mm)

L_b = length between points that are either braced against lateral displacement of the compression region or braced against twist of the cross section, in. (mm)

(c) For rounds and rectangular bars bent about their minor axis, the *limit state* of *lateral-torsional buckling* need not be considered.

F12. UNSYMMETRICAL SHAPES

This section applies to all unsymmetrical shapes, except single angles.

The *nominal flexural strength*, M_n, shall be the lowest value obtained according to the *limit states* of *yielding (yield moment), lateral-torsional buckling* and *local buckling* where

$$M_n = F_n S \tag{F12-1}$$

where

S = lowest elastic section modulus relative to the axis of bending, in.^{3} (mm^3)

1. Yielding

$$F_n = F_y \tag{F12-2}$$

2. Lateral-Torsional Buckling

$$F_n = F_{cr} \leq F_y \tag{F12-3}$$

where

F_{cr} = buckling *stress* for the section as determined by analysis, ksi (MPa)

User Note: In the case of Z-shaped members, it is recommended that F_{cr} be taken as $0.5F_{cr}$ of a channel with the same flange and web properties.

3. Local Buckling

$$F_n = F_{cr} \leq F_y \tag{F12-4}$$

where

F_{cr} = buckling *stress* for the section as determined by analysis, ksi (MPa)

F13. PROPORTIONS OF BEAMS AND GIRDERS

1. Hole Reductions

This section applies to rolled or built-up shapes, and cover-plated *beams* with holes, proportioned on the basis of flexural strength of the gross section.

In addition to the *limit states* specified in other sections of this Chapter, the *nominal flexural strength*, M_n, shall be limited according to the limit state of *tensile rupture* of the tension flange.

(a) For $F_u A_{fn} \geq Y_t F_y A_{fg}$, the limit state of tensile rupture does not apply.

(b) For $F_u A_{fn} < Y_t F_y A_{fg}$, the nominal flexural strength, M_n, at the location of the holes in the tension flange shall not be taken greater than:

$$M_n = \frac{F_u A_{fn}}{A_{fg}} S_x \tag{F13-1}$$

where

A_{fg} = gross tension flange area, calculated in accordance with the provisions of Section D3.1, in.^{2} (mm^2)

A_{fn} = net tension flange area, calculated in accordance with the provisions of Section D3.2, in.^{2} (mm^2)

Y_t = 1.0 for $F_y/F_u \leq 0.8$
= 1.1 otherwise

2. Proportioning Limits for I-Shaped Members

Singly symmetric I-shaped members shall satisfy the following limit:

$$0.1 \leq \frac{I_{yc}}{I_y} \leq 0.9 \tag{F13-2}$$

I-shaped members with slender webs shall also satisfy the following limits:

(a) For $\frac{a}{h} \leq 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = 11.7\sqrt{\frac{E}{F_y}} \tag{F13-3}$$

(b) For $\frac{a}{h} > 1.5$

$$\left(\frac{h}{t_w}\right)_{max} = \frac{0.42E}{F_y} \tag{F13-4}$$

where

a = clear distance between *transverse stiffeners*, in. (mm)

In unstiffened girders h/t_w shall not exceed 260. The ratio of the web area to the compression flange area shall not exceed 10.

3. Cover Plates

Flanges of welded *beams* or girders may be varied in thickness or width by splicing a series of plates or by the use of *cover plates*.

The total cross-sectional area of cover plates of bolted girders shall not exceed 70 percent of the total flange area.

High-strength bolts or welds connecting flange to web, or cover plate to flange, shall be proportioned to resist the total *horizontal shear* resulting from the bending *force*s on the girder. The longitudinal distribution of these bolts or intermittent welds shall be in proportion to the intensity of the shear.

However, the longitudinal spacing shall not exceed the maximum permitted for compression or tension members in Section E6 or D4, respectively. Bolts or welds connecting flange to web shall also be proportioned to transmit to the web any *loads* applied directly to the flange, unless provision is made to transmit such loads by direct bearing.

Partial-length cover plates shall be extended beyond the theoretical cutoff point and the extended portion shall be attached to the beam or girder by high-strength bolts in a slip-critical *connection* or *fillet welds*. The attachment shall be adequate, at the applicable strength given in Sections J2.2, J3.8, or B3.9 to develop the cover plate's portion of the flexural strength in the beam or girder at the theoretical cutoff point.

For welded cover plates, the welds connecting the cover plate termination to the beam or girder shall have continuous welds along both edges of the cover plate in the length a', defined below, and shall be adequate to develop the cover plate's portion of the strength of the beam or girder at the distance a' from the end of the cover plate.

(a) When there is a continuous weld equal to or larger than three-fourths of the plate thickness across the end of the plate

$$a' = w \tag{F13-5}$$

where

w = width of cover plate, in. (mm)

(b) When there is a continuous weld smaller than three-fourths of the plate thickness across the end of the plate

$$a' = 1.5w \tag{F13-6}$$

(c) When there is no weld across the end of the plate

$$a' = 2w \tag{F13-7}$$

4. Built-Up Beams

Where two or more *beams* or channels are used side-by-side to form a flexural member, they shall be connected together in compliance with Section E6.2. When concentrated *loads* are carried from one beam to another, or distributed between the beams, *diaphragms* having sufficient *stiffness* to distribute the load shall be welded or bolted between the beams.

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

This chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and *HSS* sections, and shear in the weak direction of singly or doubly symmetric shapes.

The chapter is organized as follows:

G1. General Provisions
G2. Members with Unstiffened or Stiffened Webs
G3. Tension Field Action
G4. Single Angles
G5. Rectangular HSS and Box Members
G6. Round HSS
G7. Weak Axis Shear in Singly and Doubly Symmetric Shapes
G8. Beams and Girders with Web Openings

User Note: For applications not included in this chapter, the following sections apply:

- H3.3 Unsymmetric sections.
- J4.2 Shear strength of connecting elements.
- J10.6 Web panel zone shear.

G1. GENERAL PROVISIONS

Two methods of calculating shear strength are presented below. The method presented in Section G2 does not utilize the post *buckling strength* of the member (*tension field action*). The method presented in Section G3 utilizes tension field action.

The *design shear strength*, $\phi_v V_n$, and the *allowable shear strength*, V_n/Ω_v, shall be determined as follows.

For all provisions in this chapter except Section G2.1a:

$$\phi_v = 0.90\,(\text{LRFD}) \qquad \Omega_v = 1.67\,(\text{ASD})$$

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

1. Nominal Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, V_n, of unstiffened or stiffened webs, according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6F_y A_w C_v \tag{G2-1}$$

(a) For webs of rolled I-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$:

$$\phi_v = 1.00\,(\text{LRFD}) \qquad \Omega_v = 1.50\,(\text{ASD})$$

and

$$C_v = 1.0 \tag{G2-2}$$

User Note: All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for $F_y \leq 50$ ksi (345 MPa).

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, C_v, is determined as follows:

(i) For $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \tag{G2-3}$$

(ii) For $1.10\sqrt{k_v E/F_y} < h/t_w \leq 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{G2-4}$$

(iii) For $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.51Ek_v}{(h/t_w)^2 F_y} \tag{G2-5}$$

where

A_w = the overall depth times the web thickness, dt_w, in.^{2} (mm^2)

The web plate buckling coefficient, k_v, is determined as follows:

(i) For unstiffened webs with $h/t_w < 260$, $k_v = 5$ except for the stem of tee shapes where $k_v = 1.2$.

(ii) For stiffened webs,

$$k_v = 5 + \frac{5}{(a/h)^2}$$

$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{(h/t_w)}\right]^2$$

where

a = clear distance between transverse *stiffener*s, in. (mm)

h = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)

= for built-up welded sections, the clear distance between flanges, in. (mm)
= for built-up bolted sections, the distance between *fastener* lines, in. (mm)
= for tees, the overall depth, in. (mm)

User Note: For all ASTM A6 W, S, M and HP shapes except M12.5×12.4, M12.5×11.6, M12×11.8, M12×10.8, M12×10, M10×8, and M10×7.5, when $F_y \leq 50$ ksi (345 MPa), $C_v = 1.0$.

2. Transverse Stiffeners

Transverse *stiffener*s are not required where $h/t_w \leq 2.46\sqrt{E/F_y}$, or where the required shear strength is less than or equal to the available shear strength provided in accordance with Section G2.1 for $k_v = 5$.

Transverse stiffeners used to develop the available web shear strength, as provided in Section G2.1, shall have a moment of inertia about an axis in the web center for *stiffener* pairs or about the face in contact with the web plate for single stiffeners, which shall not be less than $at_w^3 j$, where

$$j = \frac{2.5}{(a/h)^2} - 2 \geq 0.5 \tag{G2-6}$$

Transverse stiffeners are permitted to be stopped short of the tension flange, provided bearing is not needed to transmit a concentrated load or reaction. The weld by which transverse stiffeners are attached to the web shall be terminated not less than four times nor more than six times the web thickness from the near toe to the web-to-flange weld. When single stiffeners are used, they shall be attached to the compression flange, if it consists of a rectangular plate, to resist any uplift tendency due to torsion in the flange. When *lateral bracing* is attached to a stiffener, or a pair of stiffeners, these, in turn, shall be connected to the compression flange to transmit 1 percent of the total flange *force*, unless the flange is composed only of angles.

Bolts connecting stiffeners to the girder web shall be spaced not more than 12 in. (305 mm) on center. If intermittent *fillet weld*s are used, the clear distance between welds shall not be more than 16 times the web thickness nor more than 10 in. (250 mm).

G3. TENSION FIELD ACTION

1. Limits on the Use of Tension Field Action

Consideration of *tension field action* is permitted for flanged members when the web plate is supported on all four sides by flanges or *stiffener*s. Consideration of tension field action is not permitted for:

(a) *end panels* in all members with transverse stiffeners;
(b) members when a/h exceeds 3.0 or $[260/(h/t_w)]^2$;
(c) $2A_w/(A_{fc} + A_{ft}) > 2.5$; or
(d) h/b_{fc} or $h/b_{ft} > 6.0$

where

A_{fc} = area of compression flange, in.^{2} (mm^2)
A_{ft} = area of tension flange, in.^{2} (mm^2)
b_{fc} = width of compression flange, in. (mm)
b_{ft} = width of tension flange, in. (mm)

In these cases, the nominal shear strength, V_n, shall be determined according to the provisions of Section G2.

2. Nominal Shear Strength with Tension Field Action

When *tension field action* is permitted according to Section G3.1, the nominal shear strength, V_n, with tension field action, according to the *limit state* of tension field *yielding*, shall be

(a) For $h/t_w \leq 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w \tag{G3-1}$$

(b) For $h/t_w > 1.10\sqrt{k_v E/F_y}$

$$V_n = 0.6F_y A_w \left(C_v + \frac{1 - C_v}{1.15\sqrt{1 + (a/h)^2}}\right) \tag{G3-2}$$

where

k_v and C_v are as defined in Section G2.1.

3. Transverse Stiffeners

Transverse *stiffeners* subject to *tension field action* shall meet the requirements of Section G2.2 and the following limitations:

(1) $(b/t)_{st} \leq 0.56\sqrt{\dfrac{E}{F_{yst}}}$

(2) $$A_{st} > \frac{F_y}{F_{yst}}\left[0.15D_s h t_w (1 - C_v)\frac{V_r}{V_c} - 18t_w^2\right] \geq 0 \tag{G3-3}$$

where

$(b/t)_{st}$ = the width-thickness ratio of the stiffener
F_{yst} = *specified minimum yield stress* of the stiffener material, ksi (MPa)
C_v = coefficient defined in Section G2.1
D_s = 1.0 for stiffeners in pairs
= 1.8 for single angle stiffeners
= 2.4 for single plate stiffeners
V_r = *required shear strength* at the location of the stiffener, kips (N)
V_c = *available shear strength*; $\phi_v V_n$ (LRFD) or V_n/Ω_v (ASD) with V_n as defined in Section G3.2, kips (N)

G4. SINGLE ANGLES

The nominal shear strength, V_n, of a single angle leg shall be determined using Equation G2-1 with $C_v = 1.0$, $A_w = bt$ where b = width of the leg resisting the shear *force*, in. (mm) and $k_v = 1.2$.

G5. RECTANGULAR HSS AND BOX MEMBERS

The *nominal shear strength*, V_n, of rectangular *HSS* and box members shall be determined using the provisions of Section G2.1 with $A_w = 2ht$ where h for the width resisting the shear *force* shall be taken as the clear distance between the flanges less the inside corner radius on each side and $t_w = t$ and $k_v = 5$. If the corner radius is not known, h shall be taken as the corresponding outside dimension minus three times the thickness.

G6. ROUND HSS

The *nominal shear strength*, V_n, of round *HSS*, according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = F_{cr} A_g / 2 \tag{G6-1}$$

where

F_{cr} shall be the larger of

$$F_{cr} = \frac{1.60E}{\sqrt{\frac{L_v}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \tag{G6-2a}$$

and

$$F_{cr} = \frac{0.78E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \tag{G6-2b}$$

but shall not exceed $0.6F_y$

A_g = gross area of section based on design wall thickness, in.^{2} (mm^2)

D = outside diameter, in. (mm)

L_v = the distance from maximum to zero shear *force*, in. (mm)

t = *design wall thickness*, equal to 0.93 times the nominal wall thickness for ERW *HSS* and equal to the nominal thickness for SAW HSS, in. (mm)

> **User Note:** The shear buckling equations, Equations G6-2a and G6-2b, will control for *D*/*t* over 100, high strength steels, and long lengths. If the shear strength for standard sections is desired, shear yielding will usually control.

G7. WEAK AXIS SHEAR IN SINGLY AND DOUBLY SYMMETRIC SHAPES

For singly and doubly symmetric shapes loaded in the *weak axis* without torsion, the nominal shear strength, V_n, for each shear resisting element shall be determined using Equation G2-1 and Section G2.1(b) with $A_w = b_f t_f$ and $k_v = 1.2$.

User Note: For all ASTM A6 W, S, M and HP shapes, when $F_y \leq 50$ ksi (345 MPa), $C_v = 1.0$.

G8. BEAMS AND GIRDERS WITH WEB OPENINGS

The effect of all web openings on the nominal shear strength of steel and *composite beams* shall be determined. Adequate reinforcement shall be provided when the *required strength* exceeds the *available strength* of the member at the opening.

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

This chapter addresses members subject to axial *force* and flexure about one or both axes, with or without torsion, and to members subject to torsion only.

The chapter is organized as follows:

H1. Doubly and Singly Symmetric Members Subject to Flexure and Axial Force
H2. Unsymmetric and Other Members Subject to Flexure and Axial Force
H3. Members under Torsion and Combined Torsion, Flexure, Shear and/or Axial Force

User Note: For *composite* members, see Chapter I.

H1. DOUBLY AND SINGLY SYMMETRIC MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

1. Doubly and Singly Symmetric Members in Flexure and Compression

The interaction of flexure and compression in doubly symmetric members and singly symmetric members for which $0.1 \le (I_{yc}/I_y) \le 0.9$, that are constrained to bend about a *geometric axis* (x and/or y) shall be limited by Equations H1-1a and H1-1b, where I_{yc} is the moment of inertia about the y-axis referred to the compression flange, in.^{4} (mm^4).

User Note: Section H2 is permitted to be used in lieu of the provisions of this section.

(a) For $\dfrac{P_r}{P_c} \ge 0.2$

$$\frac{P_r}{P_c} + \frac{8}{9}\left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0 \qquad \text{(H1-1a)}$$

(b) For $\dfrac{P_r}{P_c} < 0.2$

$$\frac{P_r}{2P_c} + \left(\frac{M_{rx}}{M_{cx}} + \frac{M_{ry}}{M_{cy}}\right) \le 1.0 \qquad \text{(H1-1b)}$$

where

P_r = required axial compressive strength, kips (N)
P_c = *available axial compressive strength*, kips (N)
M_r = *required flexural strength*, kip-in. (N-mm)

M_c = *available flexural strength*, kip-in. (N-mm)
x = subscript relating symbol to *strong axis* bending
y = subscript relating symbol to *weak axis* bending

For design according to Section B3.3 (LRFD)

P_r = *required axial compressive strength* using *LRFD load combinations*, kips (N)
$P_c = \phi_c P_n$ = *design axial compressive strength*, determined in accordance with Chapter E, kips (N)
M_r = *required flexural strength* using LRFD load combinations, kip-in. (N-mm)
$M_c = \phi_b M_n$ = *design flexural strength* determined in accordance with Chapter F, kip-in. (N-mm)
ϕ_c = *resistance factor* for compression = 0.90
ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.4 (ASD)

P_r = required axial compressive strength using *ASD load combinations*, kips (N)
$P_c = P_n/\Omega_c$ = *allowable axial compressive strength*, determined in accordance with Chapter E, kips (N)
M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)
$M_c = M_n/\Omega_b$ = *allowable flexural strength* determined in accordance with Chapter F, kip-in. (N-mm)
Ω_c = *safety factor* for compression = 1.67
Ω_b = safety factor for flexure = 1.67

2. Doubly and Singly Symmetric Members in Flexure and Tension

The interaction of flexure and tension in doubly symmetric members and singly symmetric members constrained to bend about a *geometric axis* (x and/or y) shall be limited by Equations H1-1a and H1-1b,

where

For design according to Section B3.3 (LRFD)

P_r = *required tensile strength* using LRFD *load combinations*, kips (N)
$P_c = \phi_t P_n$ = *design tensile strength*, determined in accordance with Section D2, kips (N)
M_r = *required flexural strength* using LRFD load combinations, kip-in. (N-mm)
$M_c = \phi_b M_n$ = *design flexural strength* determined in accordance with Chapter F, kip-in. (N-mm)
ϕ_t = *resistance factor* for tension (see Section D2)
ϕ_b = resistance factor for flexure = 0.90

For doubly symmetric members, C_b in Chapter F may be increased by $\sqrt{1+\dfrac{P_u}{P_{ey}}}$ for axial tension that acts concurrently with flexure,

where

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2}$$

For design according to Section B3.4 (ASD)

P_r = required tensile strength using *ASD load combinations*, kips (N)

P_c = P_n/Ω_t = *allowable tensile strength*, determined in accordance with Section D2, kips (N)

M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)

M_c = M_n/Ω_b = *allowable flexural strength* determined in accordance with Chapter F, kip-in. (N-mm)

Ω_t = *safety factor* for tension (see Section D2)

Ω_b = safety factor for flexure = 1.67

For doubly symmetric members, C_b in Chapter F may be increased by $\sqrt{1+\dfrac{1.5P_a}{P_{ey}}}$ for axial tension that acts concurrently with flexure

where

$$P_{ey} = \frac{\pi^2 EI_y}{L_b^2}$$

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equations H1-1a and H1-1b.

3. Doubly Symmetric Members in Single Axis Flexure and Compression

For doubly symmetric members in flexure and compression with moments primarily in one plane, it is permissible to consider the two independent *limit states*, *in-plane instability* and *out-of-plane buckling* or *flexural-torsional buckling*, separately in lieu of the combined approach provided in Section H1.1.

(a) For the limit state of in-plane instability, Equations H1-1 shall be used with P_c, M_r, and M_c determined in the plane of bending.

(b) For the limit state of out-of-plane buckling

$$\frac{P_r}{P_{co}} + \left(\frac{M_r}{M_{cx}}\right)^2 \le 1.0 \qquad \text{(H1-2)}$$

where

P_{co} = *available compressive strength* out of the plane of bending, kips (N)

M_{cx} = *available flexural-torsional strength* for *strong axis* flexure determined from Chapter F, kip-in. (N-mm)

If bending occurs only about the *weak axis*, the moment ratio in Equation H1-2 shall be neglected.

For members with significant biaxial moments ($M_r/M_c \geq 0.05$ in both directions), the provisions of Section H1.1 shall be followed.

H2. UNSYMMETRIC AND OTHER MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

This section addresses the interaction of flexure and axial *stress* for shapes not covered in Section H1. It is permitted to use the provisions of this Section for any shape in lieu of the provisions of Section H1.

$$\left| \frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \right| \leq 1.0 \qquad \text{(H2-1)}$$

where

f_a = required axial stress at the point of consideration, ksi (MPa)
F_a = *available axial stress* at the point of consideration, ksi (MPa)
f_{bw}, f_{bz} = required flexural stress at the point of consideration, ksi (MPa)
F_{bw}, F_{bz} = *available flexural stress* at the point of consideration, ksi (MPa)
w = subscript relating symbol to major principal axis bending
z = subscript relating symbol to minor principal axis bending

For design according to Section B3.3 (LRFD)

f_a = required axial stress using *LRFD load combinations*, ksi (MPa)
F_a = $\phi_c F_{cr}$ = *design axial stress*, determined in accordance with Chapter E for compression or Section D2 for tension, ksi (MPa)
f_{bw}, f_{bz} = required flexural *stress* at the specific location in the cross section using LRFD load combinations, ksi (MPa)
$F_{bw}, F_{bz} = \dfrac{\phi_b M_n}{S}$ = *design flexural stress* determined in accordance with Chapter F, ksi (MPa). Use the section modulus for the specific location in the cross section and consider the sign of the stress.
ϕ_c = *resistance factor* for compression = 0.90
ϕ_t = resistance factor for tension (Section D2)
ϕ_b = resistance factor for flexure = 0.90

For design according to Section B3.4 (ASD)

f_a = required axial stress using ASD *load combinations*, ksi (MPa)
$F_a = \dfrac{F_{cr}}{\Omega_c}$ = *allowable axial stress* determined in accordance with Chapter E for compression, or Section D2 for tension, ksi (MPa)
f_{bw}, f_{bz} = required flexural stress at the specific location in the cross section using *ASD load combinations*, ksi (MPa)

$F_{bw}, F_{bz} = \dfrac{M_n}{\Omega_b S}$ = *allowable flexural stress* determined in accordance with Chapter F, ksi (MPa). Use the section modulus for the specific location in the cross section and consider the sign of the stress.

Ω_c = *safety factor* for compression = 1.67

Ω_t = safety factor for tension (Section D2)

Ω_b = safety factor for flexure = 1.67

Equation H2-1 shall be evaluated using the principal bending axes by considering the sense of the flexural stresses at the critical points of the cross section. The flexural terms are either added to or subtracted from the axial term as appropriate. When the axial *force* is compression, second order effects shall be included according to the provisions of Chapter C.

A more detailed analysis of the interaction of flexure and tension is permitted in lieu of Equation H2-1.

H3. MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR AND/OR AXIAL FORCE

1. Torsional Strength of Round and Rectangular HSS

The *design torsional strength*, $\phi_T T_n$, and the *allowable torsional strength*, T_n/Ω_T, for round and rectangular *HSS* shall be determined as follows:

$$\phi_T = 0.90 \text{ (LRFD)} \qquad \Omega_T = 1.67 \text{ (ASD)}$$

The nominal torsional strength, T_n, according to the *limit states* of *torsional yielding* and *torsional buckling* is:

$$T_n = F_{cr} C \tag{H3-1}$$

where

C is the *HSS* torsional constant

F_{cr} shall be determined as follows:

(a) For round *HSS*, F_{cr} shall be the larger of

$$F_{cr} = \frac{1.23E}{\sqrt{\frac{L}{D}}\left(\frac{D}{t}\right)^{\frac{5}{4}}} \tag{H3-2a}$$

and

$$F_{cr} = \frac{0.60E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \tag{H3-2b}$$

but shall not exceed $0.6F_y$,

where

L = length of the member, in. (mm)

D = outside diameter, in. (mm)

(b) For rectangular *HSS*

(i) For $h/t \leq 2.45\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y \tag{H3-3}$$

(ii) For $2.45\sqrt{E/F_y} < h/t \leq 3.07\sqrt{E/F_y}$

$$F_{cr} = 0.6F_y(2.45\sqrt{E/F_y})/(h/t) \tag{H3-4}$$

(iii) For $3.07\sqrt{E/F_y} < h/t \leq 260$

$$F_{cr} = 0.458\pi^2 E/(h/t)^2 \tag{H3-5}$$

User Note: The torsional shear constant, C, may be conservatively taken as:

For a round *HSS*: $C = \dfrac{\pi(D-t)^2 t}{2}$

For rectangular *HSS*: $C = 2(B-t)(H-t)t - 4.5(4-\pi)t^3$

2. HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

When the *required torsional strength*, T_r, is less than or equal to 20 percent of the *available torsional strength*, T_c, the interaction of torsion, shear, flexure and/or axial *force* for *HSS* shall be determined by Section H1 and the torsional effects shall be neglected. When T_r exceeds 20 percent of T_c, the interaction of torsion, shear, flexure and/or axial force shall be limited by

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0 \tag{H3-6}$$

where

For design according to Section B3.3 (LRFD)

P_r = *required axial strength* using *LRFD load combinations*, kips (N)

P_c = ϕP_n, *design tensile* or *compressive strength* in accordance with Chapter D or E, kips (N)

M_r = *required flexural strength* using LRFD load combinations, kip-in. (N-mm)

M_c = $\phi_b M_n$, *design flexural strength* in accordance with Chapter F, kip-in. (N-mm)

V_r = *required shear strength* using LRFD load combinations, kips (N)

V_c = $\phi_v V_n$, *design shear strength* in accordance with Chapter G, kips (N)

T_r = *required torsional strength* using LRFD load combinations, kip-in. (N-mm)

T_c = $\phi_T T_n$, *design torsional strength* in accordance with Section H3.1, kip-in. (N-mm)

For design according to Section B3.4 (ASD)

P_r = required axial strength using *ASD load combinations*, kips (N)

P_c = P_n/Ω, *allowable tensile* or *compressive strength* in accordance with Chapter D or E, kips (N)

M_r = required flexural strength using ASD load combinations determined in accordance with Section B5, kip-in. (N-mm)

M_c = M_n/Ω_b, *allowable flexural strength* in accordance with Chapter F, kip-in. (N-mm)

V_r = required shear strength using ASD load combinations, kips (N)

V_c = V_n/Ω_v, *allowable shear strength* in accordance with Chapter G, kips (N)

T_r = required torsional strength using ASD load combinations, kip-in. (N-mm)

T_c = T_n/Ω_T, *allowable torsional strength* in accordance with Section H3.1, kip-in. (N-mm)

3. Strength of Non-HSS Members under Torsion and Combined Stress

The *design torsional strength*, $\phi_T F_n$, and the *allowable torsional strength*, F_n/Ω_T, for non-*HSS* members shall be the lowest value obtained according to the *limit states* of *yielding* under normal *stress*, *shear yielding* under shear stress, or *buckling*, determined as follows:

$$\phi_T = 0.90\,(\text{LRFD}) \qquad \Omega_T = 1.67\,(\text{ASD})$$

(a) For the limit state of yielding under normal stress

$$F_n = F_y \tag{H3-7}$$

(b) For the limit state of shear yielding under shear stress

$$F_n = 0.6F_y \tag{H3-8}$$

(c) For the limit state of buckling

$$F_n = F_{cr} \tag{H3-9}$$

where

F_{cr} = buckling stress for the section as determined by analysis, ksi (MPa)

Some constrained local yielding is permitted adjacent to areas that remain elastic.

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

This chapter addresses *composite columns* composed of rolled or built-up structural steel shapes or *HSS*, and structural concrete acting together, and steel *beams* supporting a reinforced concrete slab so interconnected that the beams and the slab act together to resist bending. Simple and continuous *composite beams* with *shear connectors* and *concrete-encased beams*, constructed with or without temporary shores, are included.

The chapter is organized as follows:

I1. General Provisions
I2. Axial Members
I3. Flexural Members
I4. Combined Axial Force and Flexure
I5. Special Cases

I1. GENERAL PROVISIONS

In determining *load effects* in members and *connections* of a structure that includes *composite* members, consideration shall be given to the effective sections at the time each increment of *load* is applied. The design, detailing and material properties related to the concrete and reinforcing steel portions of composite construction shall comply with the reinforced concrete and reinforcing bar design specifications stipulated by the *applicable building code*. In the absence of a building code, the provisions in ACI 318 shall apply.

1. Nominal Strength of Composite Sections

Two methods are provided for determining the *nominal strength* of *composite* sections: the *plastic stress distribution method* and the *strain-compatibility method.*

The *tensile strength* of the concrete shall be neglected in the determination of the nominal strength of composite members.

1a. Plastic Stress Distribution Method

For the plastic stress distribution method, the nominal strength shall be computed assuming that steel components have reached a *stress* of F_y in either tension or compression and concrete components in compression have reached a *stress* of $0.85f_c'$. For round *HSS* filled with concrete, a *stress* of $0.95f_c'$ is permitted to be used for concrete components in uniform compression to account for the effects of concrete confinement.

1b. Strain-Compatibility Method

For the strain compatibility method, a linear distribution of strains across the section shall be assumed, with the maximum concrete compressive strain equal to 0.003 in./in. (mm/mm). The stress-strain relationships for steel and concrete shall be obtained from tests or from published results for similar materials.

> **User Note**: The strain compatibility method should be used to determine nominal strength for irregular sections and for cases where the steel does not exhibit elasto-plastic behavior. General guidelines for the strain-compatibility method for encased columns are given in AISC Design Guide 6 and ACI 318 Sections 10.2 and 10.3.

2. Material Limitations

Concrete and steel reinforcing bars in *composite* systems shall be subject to the following limitations.

(1) For the determination of the *available strength*, concrete shall have a compressive strength f_c' of not less than 3 ksi (21 MPa) nor more than 10 ksi (70 MPa) for normal weight concrete and not less than 3 ksi (21 MPa) nor more than 6 ksi (42 MPa) for lightweight concrete.

> **User Note:** Higher strength concrete materials may be used for *stiffness* calculations but may not be relied upon for strength calculations unless justified by testing or analysis.

(2) The *specified minimum yield stress* of structural steel and reinforcing bars used in calculating the strength of a *composite column* shall not exceed 75 ksi (525 MPa).

Higher material strengths are permitted when their use is justified by testing or analysis.

> **User Note:** Additional reinforced concrete material limitations are specified in ACI 318.

3. Shear Connectors

Shear connectors shall be headed steel studs not less than four stud diameters in length after installation, or hot-rolled steel channels. Shear stud design values shall be taken as per Sections I2.1g and I3.2d(2). Stud connectors shall conform to the requirements of Section A3.6. Channel connectors shall conform to the requirements of Section A3.1.

I2. AXIAL MEMBERS

This section applies to two types of *composite* axial members: encased and filled sections.

1. Encased Composite Columns

1a. Limitations

To qualify as an encased *composite column*, the following limitations shall be met:

(1) The cross-sectional area of the steel core shall comprise at least 1 percent of the total composite cross section.

(2) Concrete encasement of the steel core shall be reinforced with continuous longitudinal bars and lateral ties or spirals. The minimum *transverse reinforcement* shall be at least 0.009 in.^{2} per in. (6 mm^2 per mm) of tie spacing.

(3) The minimum reinforcement ratio for continuous longitudinal reinforcing, ρ_{sr}, shall be 0.004, where ρ_{sr} is given by:

$$\rho_{sr} = \frac{A_{sr}}{A_g} \tag{I2-1}$$

where

A_{sr} = area of continuous reinforcing bars, in.^{2} (mm^2)
A_g = gross area of composite member, in.^{2} (mm^2)

1b. Compressive Strength

The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c, for axially loaded *encased composite columns* shall be determined for the limit state of *flexural buckling* based on column slenderness as follows:

$$\phi_c = 0.75\,(\text{LRFD}) \qquad \Omega_c = 2.00\,(\text{ASD})$$

(a) When $P_e \geq 0.44P_o$

$$P_n = P_o\left[0.658^{\left(\frac{P_o}{P_e}\right)}\right] \tag{I2-2}$$

(b) When $P_e < 0.44P_o$

$$P_n = 0.877P_e \tag{I2-3}$$

where

$$P_o = A_sF_y + A_{sr}F_{yr} + 0.85A_cf'_c \tag{I2-4}$$

$$P_e = \pi^2(EI_{eff})/(KL)^2 \tag{I2-5}$$

and where

A_s = area of the steel section, in.^{2} (mm^2)
A_c = area of concrete, in.^{2} (mm^2)
A_{sr} = area of continuous reinforcing bars, in.^{2} (mm^2)
E_c = modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f'_c}$, ksi ($0.043w_c^{1.5}\sqrt{f'_c}$, MPa)
E_s = modulus of elasticity of steel = 29,000 ksi (210 MPa)
f'_c = specified compressive strength of concrete, ksi (MPa)
F_y = *specified minimum yield stress* of steel section, ksi (MPa)
F_{yr} = specified minimum yield stress of reinforcing bars, ksi (MPa)
I_c = moment of inertia of the concrete section, in.^{4} (mm^4)
I_s = moment of inertia of steel shape, in.^{4} (mm^4)
I_{sr} = moment of inertia of reinforcing bars, in.^{4} (mm^4)

K = the effective length factor determined in accordance with Chapter C
L = laterally unbraced length of the member, in. (mm)
w_c = weight of concrete per unit volume ($90 \leq w_c \leq 155$ lbs/ft^3 or $1500 \leq w_c \leq 2500$ kg/m^3)

where

EI_{eff} = effective stiffness of composite section, kip-in.^{2} (N-mm^2)

$$EI_{eff} = E_s I_s + 0.5 E_s I_{sr} + C_1 E_c I_c \quad \text{(I2-6)}$$

where

$$C_1 = 0.1 + 2\left(\frac{A_s}{A_c + A_s}\right) \leq 0.3 \quad \text{(I2-7)}$$

1c. Tensile Strength

The *design tensile strength*, $\phi_t P_n$, and *allowable tensile strength*, P_n/Ω_t, for encased composite columns shall be determined for the limit state of *yielding* as

$$P_n = A_s F_y + A_{sr} F_{yr} \quad \text{(I2-8)}$$

$$\phi_t = 0.90\,\text{(LRFD)} \qquad \Omega_t = 1.67\,\text{(ASD)}$$

1d. Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone as specified in Chapter G plus the shear strength provided by tie reinforcement, if present, or the shear strength of the reinforced concrete portion alone.

User Note: The nominal shear strength of tie reinforcement may be determined as $A_{st} F_{yr}(d/s)$ where A_{st} is the area of tie reinforcement, d is the effective depth of the concrete section, and s is the spacing of the tie reinforcement. The shear capacity of reinforced concrete may be determined according to ACI 318, Chapter 11.

1e. Load Transfer

*Load*s applied to axially loaded encased composite columns shall be transferred between the steel and concrete in accordance with the following requirements:

(a) When the external *force* is applied directly to the steel section, *shear connectors* shall be provided to transfer the required shear force, V', as follows:

$$V' = V(1 - A_s F_y / P_o) \quad \text{(I2-9)}$$

where

V = required shear force introduced to *column*, kips (N)
A_s = area of steel cross section, in.^{2} (mm^2)
P_o = nominal axial compressive strength without consideration of *length effects*, kips (N)

(b) When the external force is applied directly to the concrete encasement, shear connectors shall be provided to transfer the required shear force, V', as follows:

$$V' = V(A_s F_y / P_o) \quad \text{(I2-10)}$$

(c) When load is applied to the concrete of an encased composite column by direct bearing the *design bearing strength*, $\phi_B P_p$, and the *allowable bearing strength*, P_p/Ω_B, of the concrete shall be:

$$P_p = 1.7 f_c' A_B \tag{I2-11}$$

$$\phi_B = 0.65 \text{ (LRFD)} \qquad \Omega_B = 2.31 \text{ (ASD)}$$

where

A_B = loaded area of concrete, in.^{2} (mm^2)

1f. Detailing Requirements

At least four continuous longitudinal reinforcing bars shall be used in encased composite columns. *Transverse reinforcement* shall be spaced at the smallest of 16 longitudinal bar diameters, 48 tie bar diameters or 0.5 times the least dimension of the composite section. The encasement shall provide at least 1.5 in. (38 mm) of clear cover to the reinforcing steel.

Shear connectors shall be provided to transfer the required shear *force* specified in Section I2.1e. The shear connectors shall be distributed along the length of the member at least a distance of 2.5 times the depth of the encased composite column above and below the load transfer region. The maximum connector spacing shall be 16 in. (405 mm). Connectors to transfer axial load shall be placed on at least two faces of the steel shape in a configuration symmetrical about the steel shape axes.

If the composite cross section is built up from two or more encased steel shapes, the shapes shall be interconnected with *lacing*, *tie plates*, *batten plates* or similar components to prevent buckling of individual shapes due to loads applied prior to hardening of the concrete.

1g. Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete is:

$$Q_n = 0.5 A_{sc} \sqrt{f_c' E_c} \leq A_{sc} F_u \tag{I2-12}$$

where

A_{sc} = cross-sectional area of stud shear connector, in.^{2} (mm^2)
F_u = *specified minimum tensile strength* of a stud shear connector, ksi (MPa)

2. Filled Composite Columns

2a. Limitations

To qualify as a filled *composite column* the following limitations shall be met:

(1) The cross-sectional area of the steel *HSS* shall comprise at least 1 percent of the total *composite* cross section.
(2) The maximum b/t ratio for a rectangular HSS used as a composite column shall be equal to $2.26\sqrt{E/F_y}$. Higher ratios are permitted when their use is justified by testing or analysis.

(3) The maximum D/t ratio for a round HSS filled with concrete shall be $0.15\ E/F_y$. Higher ratios are permitted when their use is justified by testing or analysis.

2b. Compressive Strength

The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c, for axially loaded filled composite columns shall be determined for the *limit state* of *flexural buckling* based on Section I2.1b with the following modifications:

$$P_o = A_s F_y + A_{sr} F_{yr} + C_2 A_c f_c' \qquad \text{(I2-13)}$$

$C_2 = 0.85$ for rectangular sections and 0.95 for circular sections

$$EI_{eff} = E_s I_s + E_s I_{sr} + C_3 E_c I_c \qquad \text{(I2-14)}$$

$$C_3 = 0.6 + 2\left(\frac{A_s}{A_c + A_s}\right) \le 0.9 \qquad \text{(I2-15)}$$

2c. Tensile Strength

The *design tensile strength*, $\phi_t P_n$, and *allowable tensile strength*, P_n/Ω_t, for filled composite columns shall be determined for the limit state of *yielding* as:

$$P_n = A_s F_y + A_{sr} F_{yr} \qquad \text{(I2-16)}$$

$$\phi_t = 0.90\,(\text{LRFD}) \qquad \Omega_t = 1.67\,(\text{ASD})$$

2d. Shear Strength

The *available shear strength* shall be calculated based on either the shear strength of the steel section alone as specified in Chapter G or the shear strength of the reinforced concrete portion alone.

User Note: The shear strength of reinforced concrete may be determined by ACI 318, Chapter 11.

2e. Load Transfer

Loads applied to filled composite columns shall be transferred between the steel and concrete. When the external force is applied either to the steel section or to the concrete infill, transfer of force from the steel section to the concrete core is required from *direct bond interaction*, shear *connection* or direct bearing. The force transfer *mechanism* providing the largest *nominal strength* may be used. These force transfer mechanisms shall not be superimposed.

When load is applied to the concrete of an encased or filled composite column by direct bearing the *design bearing strength*, $\phi_B P_p$, and the *allowable bearing strength*, P_p/Ω_B, of the concrete shall be:

$$P_p = 1.7 f_c' A_B \qquad \text{(I2-17)}$$

$$\phi_B = 0.65\,(\text{LRFD}) \qquad \Omega_B = 2.31\,(\text{ASD})$$

where

A_B is the loaded area, in.^{2} (mm^2)

2f. Detailing Requirements

Where required, shear connectors transferring the required shear force shall be distributed along the length of the member at least a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS both above and below the load transfer region. The maximum connector spacing shall be 16 in. (405 mm).

I3. FLEXURAL MEMBERS

1. General

1a. Effective Width

The *effective width* of the concrete slab is the sum of the effective widths for each side of the *beam* centerline, each of which shall not exceed:

(1) one-eighth of the beam span, center-to-center of supports;
(2) one-half the distance to the centerline of the adjacent beam; or
(3) the distance to the edge of the slab.

1b. Shear Strength

The available shear strength of *composite beams* with *shear connectors* shall be determined based upon the properties of the steel section alone in accordance with Chapter G. The available shear strength of concrete-encased and filled *composite* members shall be determined based upon the properties of the steel section alone in accordance with Chapter G or based upon the properties of the concrete and longitudinal steel reinforcement.

User Note: The shear strength of the reinforced concrete may be determined in accordance with ACI 318, Chapter 11.

1c. Strength During Construction

When temporary shores are not used during construction, the steel section alone shall have adequate strength to support all *load*s applied prior to the concrete attaining 75 percent of its specified strength f_c'. The available flexural strength of the steel section shall be determined according to Chapter F.

2. Strength of Composite Beams with Shear Connectors

2a. Positive Flexural Strength

The *design positive flexural strength*, $\phi_b M_n$, and the *allowable positive flexural strength*, M_n/Ω_b, shall be determined for the *limit state* of *yielding* as follows:

$$\phi_b = 0.90\,\text{(LRFD)} \qquad \Omega_b = 1.67\,\text{(ASD)}$$

(a) For $h/t_w \leq 3.76\sqrt{E/F_y}$,

M_n shall be determined from the plastic *stress* distribution on the *composite* section for the limit state of *yielding* (*plastic moment*).

> **User Note:** All current ASTM A6 W, S and HP shapes satisfy the limit given in Section I3.2a(a) for $F_y \leq 50$ ksi (345 MPa).

(b) For $h/t_w > 3.76\sqrt{E/F_y}$,

M_n shall be determined from the superposition of elastic stresses, considering the effects of shoring, for the limit state of *yielding* (*yield moment*).

2b. Negative Flexural Strength

The *design negative flexural strength*, $\phi_b M_n$, and *the allowable negative flexural strength*, M_n/Ω_b, shall be determined for the steel section alone, in accordance with the requirements of Chapter F.

Alternatively, the available negative flexural strength shall be determined from the plastic stress distribution on the *composite* section, for the *limit state* of *yielding (plastic moment),* with

$$\phi_b = 0.90\,(\text{LRFD}) \qquad \Omega_b = 1.67\,(\text{ASD})$$

provided that:

(1) The steel *beam* is *compact* and is adequately braced according to Chapter F.
(2) *Shear connectors* connect the slab to the steel beam in the negative moment region.
(3) The slab reinforcement parallel to the steel beam, within the *effective width* of the slab, is *properly developed.*

2c. Strength of Composite Beams with Formed Steel Deck

(1) General

The *available flexural strength* of composite construction consisting of concrete slabs on *formed steel deck* connected to steel beams shall be determined by the applicable portions of Section I3.2a and I3.2b, with the following requirements:

(a) This section is applicable to decks with *nominal rib height* not greater than 3 in. (75 mm). The average width of concrete rib or haunch, w_r, shall be not less than 2 in. (50 mm), but shall not be taken in calculations as more than the minimum clear width near the top of the steel deck.

(b) The concrete slab shall be connected to the steel beam with welded stud shear connectors $^3/_4$ in. (19 mm) or less in diameter (AWS D1.1). Studs shall be welded either through the deck or directly to the steel cross section. Stud shear connectors, after installation, shall extend not less than $1^1/_2$ in. (38 mm) above the top of the steel deck and there shall be at least $^1/_2$ in. (13 mm) of concrete cover above the top of the installed studs.

(c) The slab thickness above the steel deck shall be not less than 2 in. (50 mm).

(d) Steel deck shall be anchored to all supporting members at a spacing not to exceed 18 in. (460 mm). Such anchorage shall be provided by stud connectors, a combination of stud connectors and arc spot (puddle) welds, or other devices specified by the designer.

(2) Deck Ribs Oriented Perpendicular to Steel Beam

Concrete below the top of the steel deck shall be neglected in determining composite section properties and in calculating A_c for deck ribs oriented perpendicular to the steel beams.

(3) Deck Ribs Oriented Parallel to Steel Beam

Concrete below the top of the steel deck may be included in determining composite section properties and shall be included in calculating A_c.

Formed steel deck ribs over supporting beams may be split longitudinally and separated to form a *concrete haunch*.

When the nominal depth of steel deck is $1^1/_2$ in. (38 mm) or greater, the average width, w_r, of the supported haunch or rib shall be not less than 2 in. (50 mm) for the first stud in the transverse row plus four stud diameters for each additional stud.

2d. Shear Connectors

(1) Load Transfer for Positive Moment

The entire *horizontal shear* at the interface between the steel *beam* and the concrete slab shall be assumed to be transferred by shear connectors, except for *concrete-encased beams* as defined in Section I3.3. For *composite* action with concrete subject to flexural compression, the total horizontal shear force, V', between the point of maximum positive moment and the point of zero moment shall be taken as the lowest value according to the *limit states* of *concrete crushing*, *tensile yielding* of the steel section, or strength of the shear connectors:

(a) Concrete crushing

$$V' = 0.85 f_c' A_c \tag{I3-1a}$$

(b) Tensile yielding of the steel section

$$V' = F_y A_s \tag{I3-1b}$$

(c) Strength of shear connectors

$$V' = \Sigma Q_n \tag{I3-1c}$$

where

A_c = area of concrete slab within *effective width*, in.^{2} (mm^2)

A_s = area of steel cross section, in.^{2} (mm^2)

ΣQ_n = sum of *nominal strengths* of shear connectors between the point of maximum positive moment and the point of zero moment, kips (N)

(2) Load Transfer for Negative Moment

In continuous *composite beams* where longitudinal reinforcing steel in the negative moment regions is considered to act compositely with the steel *beam*, the total *horizontal shear force* between the point of maximum negative moment and the point of zero moment shall be taken as the lower value according to the limit states of *yielding* of the steel reinforcement in the slab, or strength of the shear connectors:

(a) Tensile yielding of the slab reinforcement

$$V' = A_r F_{yr} \tag{I3-2a}$$

where

A_r = area of adequately developed longitudinal reinforcing steel within the *effective width* of the concrete slab, in.^{2}(mm^2)

F_{yr} = *specified minimum yield stress* of the reinforcing steel, ksi (MPa)

(b) Strength of shear connectors

$$V' = \Sigma Q_n \tag{I3-2b}$$

(3) Strength of Stud Shear Connectors

The *nominal strength* of one stud shear connector embedded in solid concrete or in a *composite* slab is

$$Q_n = 0.5 A_{sc}\sqrt{f'_c E_c} \le R_g R_p A_{sc} F_u \tag{I3-3}$$

where

A_{sc} = cross-sectional area of stud *shear connector*, in.^{2} (mm^2)

E_c = modulus of elasticity of concrete = $w_c^{1.5}\sqrt{f'_c}$, ksi ($0.043 w_c^{1.5}\sqrt{f'_c}$, MPa)

F_u = *specified minimum tensile strength* of a stud shear connector, ksi (MPa)

R_g = 1.0; (a) for one stud welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for any number of studs welded in a row directly to the steel shape; (c) for any number of studs welded in a row through steel deck with the deck oriented parallel to the steel shape and the ratio of the *average rib width* to rib depth ≥ 1.5

= 0.85; (a) for two studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape; (b) for one stud welded through steel deck with the deck oriented parallel to the steel shape and the ratio of the average rib width to rib depth <1.5

= 0.7 for three or more studs welded in a steel deck rib with the deck oriented perpendicular to the steel shape

R_p = 1.0 for studs welded directly to the steel shape (in other words, not through steel deck or sheet) and having a haunch detail with not more than 50 percent of the top flange covered by deck or sheet steel closures

= 0.75; (a) for studs welded in a *composite* slab with the deck oriented perpendicular to the *beam* and $e_{mid\text{-}ht} \ge 2$ in. (50 mm); (b) for studs welded through steel deck, or steel sheet used as *girder filler* material, and embedded in a *composite* slab with the deck oriented parallel to the *beam*

= 0.6 for studs welded in a composite slab with deck oriented perpendicular to the beam and $e_{mid\text{-}ht} < 2$ in. (50 mm)

$e_{mid\text{-}ht}$ = distance from the edge of stud shank to the steel deck web, measured at mid-height of the deck rib, and in the *load* bearing direction of the stud (in other words, in the direction of maximum moment for a simply supported beam), in. (mm)

w_c = weight of concrete per unit volume ($90 \le w_c \le 155$ lbs/ft^3 or $1500 \le w_c \le 2500$ kg/m^3)

User Note: The table below presents values for R_g and R_p for several cases.

Condition	R_g	R_p
No decking*	1.0	1.0
Decking oriented parallel to the steel shape		
$\frac{w_r}{h_r} \ge 1.5$	1.0	0.75
$\frac{w_r}{h_r} < 1.5$	0.85**	0.75
Decking oriented perpendicular to the steel shape Number of studs occupying the same decking rib		
1	1.0	0.6$^+$
2	0.85	0.6$^+$
3 or more	0.7	0.6$^+$

h_r = nominal rib height, in. (mm)

w_r = average width of concrete rib or haunch (as defined in Section I3.2c), in. (mm)

* to qualify as "no decking," stud shear connectors shall be welded directly to the steel shape and no more than 50 percent of the top flange of the steel shape may be covered by decking or sheet steel, such as girder filler material.

** for a single stud

$^+$ this value may be increased to 0.75 when $e_{mid\text{-}ht} \ge 2$ in. (51 mm)

(4) Strength of Channel Shear Connectors

The nominal strength of one channel shear connector embedded in a solid concrete slab is

$$Q_n = 0.3(t_f + 0.5t_w)L_c\sqrt{f'_c E_c} \qquad \text{(I3-4)}$$

where

t_f = flange thickness of channel shear connector, in. (mm)

t_w = web thickness of channel shear connector, in. (mm)

L_c = length of channel shear connector, in. (mm)

The strength of the channel shear connector shall be developed by welding the channel to the beam flange for a force equal to Q_n, considering eccentricity on the connector.

(5) Required Number of Shear Connectors

The number of shear connectors required between the section of maximum bending moment, positive or negative, and the adjacent section of zero moment shall be equal to the *horizontal shear force* as determined in Sections I3.2d(1) and I3.2d(2) divided by the nominal strength of one shear connector as determined from Section I3.2d(3) or Section I3.2d(4).

(6) Shear Connector Placement and Spacing

Shear connectors required on each side of the point of maximum bending moment, positive or negative, shall be distributed uniformly between that point and the adjacent points of zero moment, unless otherwise specified. However, the number of shear connectors placed between any concentrated *load* and the nearest point of zero moment shall be sufficient to develop the maximum moment required at the *concentrated load* point.

Shear connectors shall have at least 1 in. (25 mm) of lateral concrete cover, except for connectors installed in the ribs of *formed steel deck*s. The diameter of studs shall not be greater than 2.5 times the thickness of the flange to which they are welded, unless located over the web. The minimum center-to-center spacing of stud connectors shall be six diameters along the longitudinal axis of the supporting *composite beam* and four diameters transverse to the longitudinal axis of the supporting composite beam, except that within the ribs of formed steel decks oriented perpendicular to the steel beam the minimum center-to-center spacing shall be four diameters in any direction. The maximum center-to-center spacing of shear connectors shall not exceed eight times the total slab thickness nor 36 in.

3. Flexural Strength of Concrete-Encased and Filled Members

The *nominal flexural strength* of concrete-encased and filled members shall be determined using one of the following methods:

(a) The superposition of elastic *stresses* on the *composite* section, considering the effects of shoring, for the *limit state* of *yielding* (*yield moment*), where

$$\phi_b = 0.90\,(\text{LRFD}) \qquad \Omega_b = 1.67\,(\text{ASD})$$

(b) The plastic stress distribution on the steel section alone, for *the limit state* of *yielding (plastic moment)*, where

$$\phi_b = 0.90\,(\text{LRFD}) \qquad \Omega_b = 1.67\,(\text{ASD})$$

(c) If *shear connectors* are provided and the concrete meets the requirements of Section I1.2, the nominal flexural strength shall be computed based upon the plastic stress distribution on the composite section or from the strain-compatibility method, where

$$\phi_b = 0.85 \text{ (LRFD)} \qquad \Omega_b = 1.76 \text{ (ASD)}$$

I4. COMBINED AXIAL FORCE AND FLEXURE

The interaction between axial forces and flexure in composite members shall account for stability as required by Chapter C. The *design compressive strength*, $\phi_c P_n$, and *allowable compressive strength*, P_n/Ω_c, and the *design flexural strength*, $\phi_b M_n$, and *allowable flexural strength*, M_n/Ω_b, are determined as follows:

$$\phi_c = 0.75 \text{ (LRFD)} \qquad \Omega_c = 2.00 \text{ (ASD)}$$

$$\phi_b = 0.90 \text{ (LRFD)} \qquad \Omega_b = 1.67 \text{ (ASD)}$$

(1) The *nominal strength* of the cross section of a *composite* member subjected to combined axial compression and flexure shall be determined using either the *plastic stress distribution method* or the *strain-compatibility method.*

(2) To account for the influence of length effects on the axial strength of the member, the nominal axial strength of the member shall be determined by Section I2 with P_o taken as the nominal axial strength of the cross section determined in Section I4 (1) above.

I5. SPECIAL CASES

When *composite* construction does not conform to the requirements of Section I1 through Section I4, the strength of *shear connectors* and details of construction shall be established by testing.

CHAPTER J

DESIGN OF CONNECTIONS

This chapter addresses connecting elements, connectors, and the affected elements of the connected members not subject to fatigue *loads*.

The chapter is organized as follows:

J1. General Provisions
J2. Welds
J3. Bolts and Threaded Parts
J4. Affected Elements of Members and Connecting Elements
J5. Fillers
J6. Splices
J7. Bearing Strength
J8. Column Bases and Bearing on Concrete
J9. Anchor Rods and Embedments
J10. Flanges and Webs with Concentrated Forces

User Note: For cases not included in this chapter, the following sections apply:
- Chapter K. Design of HSS and Box Member Connections
- Appendix 3. Design for Fatigue

J1. GENERAL PROVISIONS

1. Design Basis

The *design strength*, ϕR_n, and the *allowable strength* R_n/Ω, of *connections* shall be determined in accordance with the provisions of this chapter and the provisions of Chapter B.

The *required strength* of the connections shall be determined by *structural analysis* for the specified *design loads*, consistent with the type of construction specified, or shall be a proportion of the required strength of the connected members when so specified herein.

Where the gravity axes of intersecting axially loaded members do not intersect at one point, the effects of eccentricity shall be considered.

2. Simple Connections

Simple connections of *beams*, girders, or trusses shall be designed as flexible and are permitted to be proportioned for the reaction shears only, except as otherwise indicated in the design documents. Flexible beam connections shall accommodate end rotations of simple beams. Some inelastic, but self-limiting deformation in the *connection* is permitted to accommodate the end rotation of a simple beam.

3. Moment Connections

End *connections* of restrained *beams*, girders, and trusses shall be designed for the combined effect of *forces* resulting from moment and shear induced by the rigidity of the connections. Response criteria for moment connections are provided in Section B3.6b.

> **User Note:** See Chapter C and Appendix 7 for analysis requirements to establish the *required strength* and *stiffness* for design of *connections*.

4. Compression Members with Bearing Joints

(a) When *column*s bear on bearing plates or are finished to bear at *splices*, there shall be sufficient connectors to hold all parts securely in place.

(b) When compression members other than columns are finished to bear, the splice material and its connectors shall be arranged to hold all parts in line and shall be proportioned for either (i) or (ii) below. It is permissible to use the less severe of the two conditions:

  (i) An axial tensile *force* of 50 percent of the required compressive strength of the member; or

  (ii) The moment and shear resulting from a transverse *load* equal to 2 percent of the required compressive strength of the member. The transverse load shall be applied at the location of the splice exclusive of other loads that act on the member. The member shall be taken as pinned for the determination of the shears and moments at the splice.

> **User Note:** All compression *joint*s should also be proportioned to resist any tension developed by the *load combinations* stipulated in Section B2.

5. Splices in Heavy Sections

When tensile *forces* due to applied tension or flexure are to be transmitted through *splice*s in heavy sections, as defined in Section A3.1c and A3.1d, by complete-joint-penetration groove (CJP) welds, material notch-toughness requirements as given in Section A3.1c and A3.1d, weld access hole details as given in Section J1.6 and thermal cut surface preparation and inspection requirements as given in M2.2 shall apply. The foregoing provision is not applicable to splices of elements of built-up shapes that are welded prior to assembling the shape.

> **User Note:** CJP groove welded splices of heavy sections can exhibit detrimental effects of weld shrinkage. Members that are sized for compression that are also subject to tensile forces may be less susceptible to damage from shrinkage if they are spliced using PJP groove welds on the flanges and fillet-welded web plates or using bolts for some or all of the splice.

6. Beam Copes and Weld Access Holes

All weld access holes required to facilitate welding operations shall have a length from the toe of the weld preparation not less than 1¹/2 times the thickness of the material in which the hole is made. The height of the access hole shall be $1^1/_2$ times the thickness of the material with the access hole, t_w, but not less than 1 in. (25 mm) nor does it need to exceed 2 in. (50 mm). The access hole shall be detailed to provide room for weld backing as needed.

For sections that are rolled or welded prior to cutting, the edge of the web shall be sloped or curved from the surface of the flange to the *reentrant* surface of the access hole. In hot-rolled shapes, and built-up shapes with CJP *groove welds* that join the web-to-flange, all *beam copes* and weld access holes shall be free of notches and sharp reentrant corners. No arc of the weld access hole shall have a radius less than $^3/_8$ in. (10 mm).

In built-up shapes with fillet or *partial-joint-penetration groove welds* that join the web-to-flange, all beam copes and weld access holes shall be free of notches and sharp reentrant corners. The access hole shall be permitted to terminate perpendicular to the flange, providing the weld is terminated at least a distance equal to the weld size away from the access hole.

For heavy sections as defined in A3.1c and A3.1d, the *thermally cut* surfaces of beam copes and weld access holes shall be ground to bright metal and inspected by either magnetic particle or dye penetrant methods prior to deposition of *splice* welds. If the curved transition portion of weld access holes and beam copes are formed by predrilled or sawed holes, that portion of the access hole or cope need not be ground. Weld access holes and beam copes in other shapes need not be ground nor inspected by dye penetrant or magnetic particle methods.

7. Placement of Welds and Bolts

Groups of welds or bolts at the ends of any member which transmit axial *force* into that member shall be sized so that the center of gravity of the group coincides with the center of gravity of the member, unless provision is made for the eccentricity. The foregoing provision is not applicable to end connections of statically loaded single angle, double angle, and similar members.

8. Bolts in Combination with Welds

Bolts shall not be considered as sharing the load in combination with welds, except that shear connections with any grade of bolts permitted by Section A3.3 installed in standard holes or short slots transverse to the direction of the load are permitted to be considered to share the load with longitudinally loaded fillet welds. In such connections the available strength of the bolts shall not be taken as greater than 50 percent of the available strength of bearing-type bolts in the connection.

In making welded alterations to structures, existing rivets and high strength bolts tightened to the requirements for *slip-critical connections* are permitted to be utilized for carrying loads present at the time of alteration and the welding need only provide the additional required strength.

9. High-Strength Bolts in Combination with Rivets

In both new work and alterations, in *connections* designed as *slip-critical connections* in accordance with the provisions of Section J3, high-strength bolts are permitted to be considered as sharing the *load* with existing rivets.

10. Limitations on Bolted and Welded Connections

*Pretensioned joint*s, slip-critical *joints* or welds shall be used for the following *connections*:

(1) *Column splice*s in all multi-story structures over 125 ft (38 m) in height
(2) Connections of all *beams* and *girders* to columns and any other beams and girders on which the bracing of columns is dependent in structures over 125 ft (38 m) in height
(3) In all structures carrying cranes of over 5-ton (50 kN) capacity: roof truss splices and connections of trusses to columns, column splices, column bracing, knee braces, and crane supports
(4) Connections for the support of machinery and other live *loads* that produce impact or reversal of load

Snug-tightened joints or joints with ASTM A307 bolts shall be permitted except where otherwise specified.

J2. WELDS

All provisions of AWS D1.1 apply under this Specification, with the exception that the provisions of the listed AISC Specification Sections apply under this Specification in lieu of the cited AWS provisions as follows:

AISC Specification Section J1.6 in lieu of AWS D1.1 Section 5.17.1
AISC Specification Section J2.2a in lieu of AWS D1.1 Section 2.3.2
AISC Specification Table J2.2 in lieu of AWS D1.1 Table 2.1
AISC Specification Table J2.5 in lieu of AWS D1.1 Table 2.3
AISC Specification Appendix 3, Table A-3.1 in lieu of AWS D1.1 Table 2.4
AISC Specification Section B3.9 and Appendix 3 in lieu of AWS D1.1 Section 2, Part C
AISC Specification Section M2.2 in lieu of AWS D1.1 Sections 5.15.4.3 and 5.15.4.4

1. Groove Welds

1a. Effective Area

The effective area of *groove welds* shall be considered as the length of the weld times the effective throat thickness.

The effective throat thickness of a *complete-joint-penetration (CJP) groove weld* shall be the thickness of the thinner part joined.

TABLE J2.1
Effective Throat of Partial-Joint-Penetration Groove Welds

Welding Process	Welding Position F (flat), H (horiz.), V (vert.), OH (overhead)	Groove Type (AWS D1.1, Figure 3.3)	Effective Throat
Shielded Metal Arc (SMAW)	All	J or U Groove 60° V	Depth of Groove
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	All		
Submerged Arc (SAW)	F	J or U Groove 60° Bevel or V	
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	F, H	45° Bevel	Depth of Groove
Shielded Metal Arc (SMAW)	All	45° Bevel	Depth of Groove Minus $^{1}/_{8}$ in. (3 mm)
Gas Metal Arc (GMAW) Flux Cored Arc (FCAW)	V, OH	45° Bevel	Depth of Groove Minus $^{1}/_{8}$ in. (3 mm)

The effective throat thickness of a *partial-joint-penetration (PJP) groove weld* shall be as shown in Table J2.1.

User Note: The effective throat size of a partial-joint-penetration groove weld is dependent on the process used and the weld position. The contract documents should either indicate the effective throat required or the weld strength required, and the fabricator should detail the *joint* based on the weld process and position to be used to weld the *joint*.

The effective weld size for flare groove welds, when filled flush to the surface of a round bar, a 90° bend in a *formed section*, or rectangular *HSS* shall be as shown in Table J2.2, unless other effective throats are demonstrated by tests. The effective size of flare groove welds filled less than flush shall be as shown in Table J2.2, less the greatest perpendicular dimension measured from a line flush to the base metal surface to the weld surface.

TABLE J2.2
Effective Weld Sizes of Flare Groove Welds

Welding Process	Flare Bevel Groove[a]	Flare V Groove
GMAW and FCAW-G	$^{5}/_{8}R$	$^{3}/_{4}R$
SMAW and FCAW-S	$^{5}/_{16}R$	$^{5}/_{8}R$
SAW	$^{5}/_{16}R$	$^{1}/_{2}R$

[a] For Flare Bevel Groove with $R < {}^{3}/_{8}$ in. (10 mm) use only reinforcing fillet weld on filled flush joint.
General Note: $R =$ radius of joint surface (can be assumed to be $2t$ for HSS), in. (mm)

TABLE J2.3
Minimum Effective Throat Thickness of Partial-Joint-Penetration Groove Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Effective Throat Thickness,[a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19) to 1 1/2 (38)	5/16 (8)
Over 1 1/2 (38) to 2 1/4 (57)	3/8 (10)
Over 2 1/4 (57) to 6 (150)	1/2 (13)
Over 6 (150)	5/8 (16)

[a] See Table J2.1.

Larger effective throat thicknesses than those in Table J2.2 are permitted, provided the fabricator can establish by qualification the consistent production of such larger effective throat thicknesses. Qualification shall consist of sectioning the weld normal to its axis, at mid-length and terminal ends. Such sectioning shall be made on a number of combinations of material sizes representative of the range to be used in the fabrication.

1b. Limitations

The minimum effective throat thickness of a partial-joint-penetration groove weld shall not be less than the size required to transmit calculated *forces* nor the size shown in Table J2.3. Minimum weld size is determined by the thinner of the two parts joined.

2. Fillet Welds

2a. Effective Area

The effective area of a *fillet weld* shall be the *effective length* multiplied by the effective throat. The effective throat of a fillet weld shall be the shortest distance from the root to the face of the diagrammatic weld. An increase in effective throat is permitted if consistent penetration beyond the root of the diagrammatic weld is demonstrated by tests using the production process and procedure variables.

For fillet welds in holes and slots, the effective length shall be the length of the centerline of the weld along the center of the plane through the throat. In the case of overlapping fillets, the effective area shall not exceed the nominal cross-sectional area of the hole or slot, in the plane of the *faying surface*.

2b. Limitations

The minimum size of fillet welds shall be not less than the size required to transmit calculated *forces*, nor the size as shown in Table J2.4. These provisions do not apply to *fillet weld reinforcements* of *partial-* or *complete-joint-penetration groove welds*.

TABLE J2.4
Minimum Size of Fillet Welds

Material Thickness of Thinner Part Joined, in. (mm)	Minimum Size of Fillet Weld,[a] in. (mm)
To 1/4 (6) inclusive	1/8 (3)
Over 1/4 (6) to 1/2 (13)	3/16 (5)
Over 1/2 (13) to 3/4 (19)	1/4 (6)
Over 3/4 (19)	5/16 (8)

[a] Leg dimension of fillet welds. Single pass welds must be used.
Note: See Section J2.2b for maximum size of fillet welds.

The maximum size of fillet welds of connected parts shall be:

(a) Along edges of material less than 1/4-in. (6 mm) thick, not greater than the thickness of the material.

(b) Along edges of material 1/4 in. (6 mm) or more in thickness, not greater than the thickness of the material minus 1/16 in. (2 mm), unless the weld is especially designated on the drawings to be built out to obtain full-throat thickness. In the as-welded condition, the distance between the edge of the base metal and the toe of the weld is permitted to be less than 1/16 in. (2 mm) provided the weld size is clearly verifiable.

The minimum effective length of fillet welds designed on the basis of strength shall be not less than four times the nominal size, or else the size of the weld shall be considered not to exceed 1/4 of its effective length. If longitudinal fillet welds are used alone in end connections of flat-bar tension members, the length of each fillet weld shall be not less than the perpendicular distance between them. For the effect of longitudinal fillet weld length in end connections upon the effective area of the connected member, see Section D3.3.

For end-loaded fillet welds with a length up to 100 times the leg dimension, it is permitted to take the effective length equal to the actual length. When the length of the end-loaded fillet weld exceeds 100 times the weld size, the effective length shall be determined by multiplying the actual length by the reduction factor, β,

$$\beta = 1.2 - 0.002(L/w) \leq 1.0 \qquad \text{(J2-1)}$$

where

L = actual length of end-loaded weld, in. (mm)
w = weld leg size, in. (mm)

When the length of the weld exceeds 300 times the leg size, the value of β shall be taken as 0.60.

Intermittent fillet welds are permitted to be used to transfer calculated *stress* across a *joint* or *faying surfaces* when the *required strength* is less than that developed by a continuous fillet weld of the smallest permitted size, and to join components of *built-up members*. The effective length of any segment of intermittent fillet welding shall be not less than four times the weld size, with a minimum of $1^1/_2$ in. (38 mm).

In *lap joints,* the minimum amount of lap shall be five times the thickness of the thinner part joined, but not less than 1 in. (25 mm). Lap joints joining plates or bars subjected to axial *stress* that utilize transverse fillet welds only shall be fillet welded along the end of both lapped parts, except where the deflection of the lapped parts is sufficiently restrained to prevent opening of the joint under maximum loading.

Fillet weld terminations are permitted to be stopped short or extend to the ends or sides of parts or be boxed except as limited by the following:

(1) For lap joints in which one connected part extends beyond an edge of another connected part that is subject to calculated tensile stress, fillet welds shall terminate not less than the size of the weld from that edge.
(2) For *connections* where flexibility of the outstanding elements is required, when *end returns* are used, the length of the return shall not exceed four times the nominal size of the weld nor half the width of the part.
(3) Fillet welds joining *transverse stiffeners* to *plate girder* webs $^3/_4$ in. (19 mm) thick or less shall end not less than four times nor more than six times the thickness of the web from the web toe of the web-to-flange welds, except where the ends of *stiffeners* are welded to the flange.
(4) Fillet welds that occur on opposite sides of a common plane, shall be interrupted at the corner common to both welds.

User Note: Fillet weld terminations should be located approximately one weld size from of the edge of the connection to minimize notches in the base metal. Fillet welds terminated at the end of the joint, other than those connecting stiffeners to girder webs, are not a cause for correction.

Fillet welds in holes or slots are permitted to be used to transmit shear in lap joints or to prevent the buckling or separation of lapped parts and to join components of built-up members. Such fillet welds may overlap, subject to the provisions of Section J2. Fillet welds in holes or slots are not to be considered plug or *slot welds*.

3. Plug and Slot Welds

3a. Effective Area

The effective shearing area of *plug* and *slot welds* shall be considered as the nominal cross-sectional area of the hole or slot in the plane of the *faying surface*.

3b. Limitations

Plug or slot welds are permitted to be used to transmit shear in *lap joints* or to prevent buckling of lapped parts and to join component parts of *built-up members*.

The diameter of the holes for a plug weld shall not be less than the thickness of the part containing it plus $5/16$ in. (8 mm), rounded to the next larger odd $1/16$ in. (even mm), nor greater than the minimum diameter plus $1/8$ in. (3 mm) or $2^1/4$ times the thickness of the weld.

The minimum center-to-center spacing of plug welds shall be four times the diameter of the hole.

The length of slot for a slot weld shall not exceed 10 times the thickness of the weld. The width of the slot shall be not less than the thickness of the part containing it plus $5/16$ in. (8 mm) rounded to the next larger odd $1/16$ in. (even mm), nor shall it be larger than $2^1/4$ times the thickness of the weld. The ends of the slot shall be semicircular or shall have the corners rounded to a radius of not less than the thickness of the part containing it, except those ends which extend to the edge of the part.

The minimum spacing of lines of slot welds in a direction transverse to their length shall be four times the width of the slot. The minimum center-to-center spacing in a longitudinal direction on any line shall be two times the length of the slot.

The thickness of plug or slot welds in material $5/8$ in. (16 mm) or less in thickness shall be equal to the thickness of the material. In material over $5/8$ in. (16 mm) thick, the thickness of the weld shall be at least one-half the thickness of the material but not less than $5/8$ in. (16 mm).

4. Strength

The *design strength*, ϕR_n and the *allowable strength*, R_n/Ω, of welds shall be the lower value of the base material and the *weld metal* strength determined according to the *limit states* of *tensile rupture*, *shear rupture* or *yielding* as follows:

For the base metal

$$R_n = F_{BM} A_{BM} \tag{J2-2}$$

For the weld metal

$$R_n = F_w A_w \tag{J2-3}$$

where

F_{BM} = *nominal strength* of the base metal per unit area, ksi (MPa)
F_w = nominal strength of the weld metal per unit area, ksi (MPa)
A_{BM} = cross-sectional area of the base metal, in.^{2} (mm^2)
A_w = effective area of the weld, in.^{2} (mm^2)

The values of ϕ, Ω, F_{BM}, and F_w and limitations thereon are given in Table J2.5.

TABLE J2.5
Available Strength of Welded Joints, kips (N)

Load Type and Direction Relative to Weld Axis	**Pertinent Metal**	**ϕ and Ω**	**Nominal Strength (F_{BM} or F_w) kips (N)**	**Effective Area (A_{BM} or A_w) in.^{2} (mm^2)**	**Required Filler Metal Strength Level[a][b]**
COMPLETE-JOINT-PENETRATION GROOVE WELDS					
Tension Normal to weld axis	Strength of the joint is controlled by the base metal				Matching filler metal shall be used. For T and corner joints with backing left in place, notch tough filler metal is required. See Section J2.6.
Compression Normal to weld axis	Strength of the joint is controlled by the base metal				Filler metal with a strength level equal to or one strength level less than matching filler metal is permitted.
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				Filler metal with a strength level equal to or less than matching filler metal is permitted.
Shear	Strength of the joint is controlled by the base metal				Matching filler metal shall be used.[c]
PARTIAL-JOINT-PENETRATION GROOVE WELDS INCLUDING FLARE VEE GROOVE AND FLARE BEVEL GROOVE WELDS					
Tension Normal to weld axis	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60 F_{EXX}$	See J2.1a	
Compression Column to Base Plate and column splices designed per J1.4(a)	Compressive stress need not be considered in design of welds joining the parts.				
Compression Connections of members designed to bear other than columns as described in J1.4(b)	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.60 F_{EXX}$	See J2.1a	
Compression Connections not finished-to-bear	Base	$\phi = 0.90$ $\Omega = 1.67$	F_y	See J4	
	Weld	$\phi = 0.80$ $\Omega = 1.88$	$0.90 F_{EXX}$	See J2.1a	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				
Shear	Base	Governed by J4			
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60 F_{EXX}$	See J2.1a	

TABLE J2.5 (cont.)
Available Strength of Welded Joints, kips (N)

Load Type and Direction Relative to Weld Axis	Pertinent Metal	ϕ and Ω	Nominal Strength (F_{bm} or F_w) kips (N)	Effective Area (A_{BM} or A_w) in.^{2} (mm^2)	Required Filler Metal Strength Level[a][b]
FILLET WELDS INCLUDING FILLETS IN HOLES AND SLOTS AND SKEWED T-JOINTS					
Shear	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$[d]	See J2.2a	
Tension or Compression Parallel to weld axis	Tension or compression in parts joined parallel to a weld need not be considered in design of welds joining the parts.				
PLUG AND SLOT WELDS					
Shear Parallel to faying surface on the effective area	Base	Governed by J4			Filler metal with a strength level equal to or less than matching filler metal is permitted.
	Weld	$\phi = 0.75$ $\Omega = 2.00$	$0.60F_{EXX}$	J2.3a	

[a] For matching weld metal see AWS D1.1, Section 3.3.

[b] Filler metal with a strength level one strength level greater than matching is permitted.

[c] Filler metals with a strength level less than matching may be used for groove welds between the webs and flanges of built-up sections transferring shear loads, or in applications where high restraint is a concern. In these applications, the weld joint shall be detailed and the weld shall be designed using the thickness of the material as the effective throat, $\phi = 0.80$, $\Omega = 1.88$ and $0.60\ F_{EXX}$ as the nominal strength.

[d] Alternatively, the provisions of J2.4(a) are permitted provided the deformation compatibility of the various weld elements is considered. Alternatively, Sections J2.4(b) and (c) are special applications of J2.4(a) that provide for deformation compatibility.

Alternatively, for *fillet weld*s loaded in-plane the *design strength*, ϕR_n and the *allowable strength*, R_n/Ω, of welds is permitted to be determined as follows:

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

(a) For a linear weld group loaded in-plane through the center of gravity

$$R_n = F_w A_w \tag{J2-4}$$

where

$$F_w = 0.60F_{EXX}\left(1.0 + 0.50\sin^{1.5}\theta\right) \tag{J2-5}$$

and

F_{EXX} = electrode classification number, ksi (MPa)
θ = angle of loading measured from the weld longitudinal axis, degrees
A_w = effective area of the weld, in.^{2} (mm^2)

User Note: A linear weld group is one in which all elements are in a line or are parallel.

(b) For weld elements within a weld group that are loaded in-plane and analyzed using an instantaneous center of rotation method, the components of the *nominal strength*, R_{nx} and R_{ny}, are permitted to be determined as follows:

$$R_{nx} = \sum F_{wix} A_{wi} \qquad R_{ny} = \sum F_{wiy} A_{wi} \tag{J2-6}$$

where

A_{wi} = effective area of weld throat of any ith weld element, in.^{2} (mm^2)

$$F_{wi} = 0.60 F_{EXX} \left(1.0 + 0.50 \sin^{1.5} \theta\right) f(p) \tag{J2-7}$$

$$f(p) = [p(1.9 - 0.9p)]^{0.3} \tag{J2-8}$$

F_{wi} = nominal *stress* in any ith weld element, ksi (MPa)

F_{wix} = x component of stress, F_{wi}

F_{wiy} = y component of stress, F_{wi}

p = Δ_i / Δ_m, ratio of element i deformation to its deformation at maximum stress

w = weld leg size, in. (mm)

r_{crit} = distance from instantaneous center of rotation to weld element with minimum Δ_u / r_i ratio, in. (mm)

Δ_i = deformation of weld elements at intermediate stress levels, linearly proportioned to the critical deformation based on distance from the instantaneous center of rotation, r_i, in. (mm)
= $r_i \Delta_u / r_{crit}$

Δ_m = $0.209(\theta + 2)^{-0.32} w$, deformation of weld element at maximum *stress*, in. (mm)

Δ_u = $1.087(\theta + 6)^{-0.65} w \leq 0.17w$, deformation of weld element at ultimate stress (fracture), usually in element furthest from instantaneous center of rotation, in. (mm)

(c) For *fillet weld* groups concentrically loaded and consisting of elements that are oriented both longitudinally and transversely to the direction of applied *load*, the combined strength, R_n, of the fillet weld group shall be determined as the greater of

$$R_n = R_{wl} + R_{wt} \tag{J2-9a}$$

or

$$R_n = 0.85 R_{wl} + 1.5 R_{wt} \tag{J2-9b}$$

where

R_{wl} = the total nominal strength of longitudinally loaded fillet welds, as determined in accordance with Table J2.5, kips (N)

R_{wt} = the total nominal strength of transversely loaded fillet welds, as determined in accordance with Table J2.5 without the alternate in Section J2.4(a), kips (N)

5. Combination of Welds

If two or more of the general types of welds (groove, fillet, plug, slot) are combined in a single *joint*, the strength of each shall be separately computed with reference to the axis of the group in order to determine the strength of the combination.

6. Filler Metal Requirements

The choice of electrode for use with *complete-joint-penetration groove welds* subject to tension normal to the effective area shall comply with the requirements for matching *filler metals* given in AWS D1.1.

> **User Note:** The following User Note Table summarizes the AWS D1.1 provisions for matching filler metals. Other restrictions exist. For a complete list of base metals and prequalified matching filler metals see AWS D1.1, Table 3.1.

Base Metal		Matching Filler Metal
A36 ≤ 3/4 in. thick		60 & 70 ksi Electrodes
A36 > 3/4 in.	A572 (Gr. 50 & 55)	SMAW: E7015, E7016, E7018, E7028
	A913 (Gr. 50)	
A588*	A992	Other processes: 70 ksi electrodes
A1011	A1018	
A913 (Gr. 60 & 65)		80 ksi electrodes

* For corrosion resistance and color similar to the base see AWS D1.1, Sect. 3.7.3

Notes:

1. Electrodes shall meet the requirements of AWS A5.1, A5.5, A5.17, A5.18, A5.20, A5.23, A5.28 and A5.29.
2. In joints with base metals of different strengths use either a filler metal that matches the higher strength base metal or a filler metal that matches the lower strength and produces a low hydrogen deposit.

Filler metal with a specified *Charpy V-Notch* (CVN) toughness of 20 ft-lbs (27 J) at 40 °F (4 °C) shall be used in the following *joints*:

(1) Complete-joint-penetration groove welded T and corner joints with steel backing left in place, subject to tension normal to the effective area, unless the joints are designed using the *nominal strength* and *resistance factor* or *safety factor* as applicable for a PJP weld.

(2) Complete-joint-penetration groove welded splices subject to tension normal to the effective area in heavy sections as defined in A3.1c and A3.1d.

The manufacturer's Certificate of Conformance shall be sufficient evidence of compliance.

7. Mixed Weld Metal

When Charpy V-Notch toughness is specified, the process consumables for all *weld metal*, tack welds, root pass and subsequent passes deposited in a *joint* shall be compatible to ensure notch-tough composite weld metal.

J3. BOLTS AND THREADED PARTS

1. High-Strength Bolts

Use of high-strength bolts shall conform to the provisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, hereafter referred to as the RCSC Specification, as approved by the Research Council on Structural Connections, except as otherwise provided in this Specification.

When assembled, all *joint* surfaces, including those adjacent to the washers, shall be free of scale, except tight *mill scale*. All ASTM A325 or A325M and A490

TABLE J3.1
Minimum Bolt Pretension, kips*

Bolt Size, in.	A325 Bolts	A490 Bolts
1/2	12	15
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1 1/8	56	80
1 1/4	71	102
1 3/8	85	121
1 1/2	103	148

*Equal to 0.70 times the minimum *tensile strength* of bolts, rounded off to nearest kip, as specified in ASTM specifications for A325 and A490 bolts with UNC threads.

TABLE J3.1M
Minimum Bolt Pretension, kN*

Bolt Size, mm	A325M Bolts	A490M Bolts
M16	91	114
M20	142	179
M22	176	221
M24	205	257
M27	267	334
M30	326	408
M36	475	595

*Equal to 0.70 times the minimum *tensile strength* of bolts, rounded off to nearest kN, as specified in ASTM specifications for A325M and A490M bolts with UNC threads.

or A490M bolts shall be tightened to a bolt tension not less than that given in Table J3.1 or J3.1M, except as noted below. Except as permitted below, installation shall be assured by any of the following methods: turn-of-nut method, a direct tension indicator, calibrated wrench or alternative design bolt.

Bolts are permitted to be installed to only the snug-tight condition when used in

(a) *bearing-type connections.*

(b) tension or combined shear and tension applications, for ASTM A325 or A325M bolts only, where loosening or *fatigue* due to vibration or *load* fluctuations are not design considerations.

The snug-tight condition is defined as the tightness attained by either a few impacts of an impact wrench or the full effort of a worker with an ordinary spud wrench that brings the connected plies into firm contact. Bolts to be tightened only to the snug-tight condition shall be clearly identified on the design and erection drawings.

When ASTM A490 or A490M bolts over 1 in. (25 mm) in diameter are used in slotted or oversized holes in external plies, a single hardened washer conforming to ASTM F436, except with 5/16-in. (8 mm) minimum thickness, shall be used in lieu of the standard washer.

TABLE J3.2
Nominal Stress of Fasteners and Threaded Parts, ksi (MPa)

Description of Fasteners	**Nominal Tensile Stress, F_{nt}, ksi (MPa)**	**Nominal Shear Stress in Bearing-Type Connections, F_{nv}, ksi (MPa)**
A307 bolts	45 (310) [a][b]	24 (165) [b][c][f]
A325 or A325M bolts, when threads are not excluded from shear planes	90 (620) [e]	48 (330) [f]
A325 or A325M bolts, when threads are excluded from shear planes	90 (620) [e]	60 (414) [f]
A490 or A490M bolts, when threads are not excluded from shear planes	113 (780) [e]	60 (414) [f]
A490 or A490M bolts, when threads are excluded from shear planes	113 (780) [e]	75 (520) [f]
Threaded parts meeting the requirements of Section A3.4, when threads are not excluded from shear planes	$0.75\ F_u$ [a][d]	$0.40F_u$
Threaded parts meeting the requirements of Section A3.4, when threads are excluded from shear planes	$0.75\ F_u$ [a][d]	$0.50F_u$

[a] Subject to the requirements of Appendix 3.

[b] For A307 bolts the tabulated values shall be reduced by 1 percent for each 1/16 in. (2 mm) over 5 diameters of length in the grip.

[c] Threads permitted in shear planes.

[d] The nominal tensile strength of the threaded portion of an upset rod, based upon the cross-sectional area at its major thread diameter, A_D, which shall be larger than the nominal body area of the rod before upsetting times F_y.

[e] For A325 or A325M and A490 or A490M bolts subject to tensile fatigue loading, see Appendix 3.

[f] When bearing-type connections used to splice tension members have a fastener pattern whose length, measured parallel to the line of force, exceeds 50 in. (1270 mm), tabulated values shall be reduced by 20 percent.

> **User Note:** Washer requirements are provided in the RCSC Specification, Section 6.

In *slip-critical connections* in which the direction of loading is toward an edge of a connected part, adequate available bearing strength shall be provided based upon the applicable requirements of Section J3.10.

When bolt requirements cannot be provided by ASTM A325 and A325M, F1852, or A490 and A490M bolts because of requirements for lengths exceeding 12 diameters or diameters exceeding 1 1/2 in. (38 mm), bolts or threaded rods conforming to ASTM A354 Gr. BC, A354 Gr. BD, or A449 are permitted to be used in accordance with the provisions for threaded rods in Table J3.2.

TABLE J3.3
Nominal Hole Dimensions, in.

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	**Oversize (Dia.)**	**Short-Slot (Width × Length)**	**Long-slot (Width × Length)**
$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{9}{16} \times \frac{11}{16}$	$\frac{9}{16} \times 1\frac{1}{4}$
$\frac{5}{8}$	$\frac{11}{16}$	$\frac{13}{16}$	$\frac{11}{16} \times \frac{7}{8}$	$\frac{11}{16} \times 1\frac{9}{16}$
$\frac{3}{4}$	$\frac{13}{16}$	$\frac{15}{16}$	$\frac{13}{16} \times 1$	$\frac{13}{16} \times 1\frac{7}{8}$
$\frac{7}{8}$	$\frac{15}{16}$	$1\frac{1}{16}$	$\frac{15}{16} \times 1\frac{1}{8}$	$\frac{15}{16} \times 2\frac{3}{16}$
1	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{1}{16} \times 1\frac{5}{16}$	$1\frac{1}{16} \times 2\frac{1}{2}$
$\geq 1\frac{1}{8}$	$d + \frac{1}{16}$	$d + \frac{5}{16}$	$(d + \frac{1}{16}) \times (d + \frac{3}{8})$	$(d + \frac{1}{16}) \times (2.5 \times d)$

TABLE J3.3M
Nominal Hole Dimensions, mm

Bolt Diameter	Hole Dimensions			
	Standard (Dia.)	**Oversize (Dia.)**	**Short-Slot (Width × Length)**	**Long-Slot (Width × Length)**
M16	18	20	18 × 22	18 × 40
M20	22	24	22 × 26	22 × 50
M22	24	28	24 × 30	24 × 55
M24	27 [a]	30	27 × 32	27 × 60
M27	30	35	30 × 37	30 × 67
M30	33	38	33 × 40	33 × 75
≥M36	$d + 3$	$d + 8$	$(d + 3) \times (d + 10)$	$(d + 3) \times 2.5d$

[a] Clearance provided allows the use of a 1-in. bolt if desirable.

When ASTM A354 Gr. BC, A354 Gr. BD, or A449 bolts and threaded rods are used in slip-critical connections, the bolt geometry including the head and nut(s) shall be equal to or (if larger in diameter) proportional to that provided by ASTM A325 and A325M, or ASTM A490 and A490M bolts. Installation shall comply with all applicable requirements of the RCSC Specification with modifications as required for the increased diameter and/ or length to provide the design pretension.

2. Size and Use of Holes

The maximum sizes of holes for bolts are given in Table J3.3 or J3.3M, except that larger holes, required for tolerance on location of anchor rods in concrete foundations, are permitted in *column* base details.

Standard holes or *short-slotted holes* transverse to the direction of the *load* shall be provided in accordance with the provisions of this specification, unless oversized holes, short-slotted holes parallel to the load or *long-slotted holes* are approved by the *engineer of record*. Finger *shims* up to $\frac{1}{4}$ in. (6 mm) are permitted in *slip-critical connections* designed on the basis of standard holes without reducing the nominal shear strength of the *fastener* to that specified for slotted holes.

Oversized holes are permitted in any or all plies of slip-critical connections, but they shall not be used in *bearing-type connections*. Hardened washers shall be installed over oversized holes in an outer ply.

Short-slotted holes are permitted in any or all plies of slip-critical or bearing-type connections. The slots are permitted without regard to direction of loading in slip-critical connections, but the length shall be normal to the direction of the load in bearing-type connections. Washers shall be installed over short-slotted holes in an outer ply; when high-strength bolts are used, such washers shall be hardened.

Long-slotted holes are permitted in only one of the connected parts of either a slip-critical or bearing-type connection at an individual *faying surface*. Long-slotted holes are permitted without regard to direction of loading in slip-critical connections, but shall be normal to the direction of load in bearing-type connections. Where long-slotted holes are used in an outer ply, plate washers, or a continuous bar with standard holes, having a size sufficient to completely cover the slot after installation, shall be provided. In high-strength bolted connections, such plate washers or continuous bars shall be not less than 5/16 in. (8 mm) thick and shall be of structural grade material, but need not be hardened. If hardened washers are required for use of high-strength bolts, the hardened washers shall be placed over the outer surface of the plate washer or bar.

3. Minimum Spacing

The distance between centers of standard, oversized, or slotted holes, shall not be less than 2 2/3 times the nominal diameter, d, of the *fastener*; a distance of $3d$ is preferred.

4. Minimum Edge Distance

The distance from the center of a standard hole to an edge of a connected part in any direction shall not be less than either the applicable value from Table J3.4 or J3.4M, or as required in Section J3.10. The distance from the center of an oversized or slotted hole to an edge of a connected part shall be not less than that required for a standard hole to an edge of a connected part plus the applicable increment C_2 from Table J3.5 or J3.5M.

User Note: The edge distances in Tables J3.4 and J3.4M are minimum edge distances based on standard fabrication practices and workmanship tolerances. The appropriate provisions of Sections J3.10 and J4 must be satisfied.

5. Maximum Spacing and Edge Distance

The maximum distance from the center of any bolt or rivet to the nearest edge of parts in contact shall be 12 times the thickness of the connected part under consideration, but shall not exceed 6 in. (150 mm). The longitudinal spacing of fasteners between elements in continuous contact consisting of a plate and a shape or two plates shall be as follows:

TABLE J3.4
Minimum Edge Distance,[a] in., from Center of Standard Hole[b] to Edge of Connected Part

Bolt Diameter (in.)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]
$^{1}/_{2}$	$^{7}/_{8}$	$^{3}/_{4}$
$^{5}/_{8}$	$1^{1}/_{8}$	$^{7}/_{8}$
$^{3}/_{4}$	$1^{1}/_{4}$	1
$^{7}/_{8}$	$1^{1}/_{2}$ [d]	$1^{1}/_{8}$
1	$1^{3}/_{4}$ [d]	$1^{1}/_{4}$
$1^{1}/_{8}$	2	$1^{1}/_{2}$
$1^{1}/_{4}$	$2^{1}/_{4}$	$1^{5}/_{8}$
Over $1^{1}/_{4}$	$1^{3}/_{4} \times$ d	$1^{1}/_{4} \times$ d

[a] Lesser edge distances are permitted to be used provided provisions of Section J3.10, as appropriate, are satisfied.

[b] For oversized or slotted holes, see Table J3.5.

[c] All edge distances in this *column* are permitted to be reduced $^{1}/_{8}$ in. when the hole is at a point where *required strength* does not exceed 25 percent of the maximum strength in the element.

[d] These are permitted to be $1^{1}/_{4}$ in. at the ends of *beam connection* angles and shear end plates.

TABLE J3.4M
Minimum Edge Distance,[a] mm, from Center of Standard Hole[b] to Edge of Connected Part

Bolt Diameter (mm)	At Sheared Edges	At Rolled Edges of Plates, Shapes or Bars, or Thermally Cut Edges [c]
16	28	22
20	34	26
22	38 [d]	28
24	42 [d]	30
27	48	34
30	52	38
36	64	46
Over 36	$1.75d$	$1.25d$

[a] Lesser edge distances are permitted to be used provided provisions of Section J3.10, as appropriate, are satisfied.

[b] For oversized or slotted holes, see Table J3.5M.

[c] All edge distances in this column are permitted to be reduced 3 mm when the hole is at a point where required strength does not exceed 25 percent of the maximum strength in the element.

[d] These are permitted to be 32 mm at the ends of beam connection angles and shear end plates.

TABLE J3.5
Values of Edge Distance Increment C_2, in.

Nominal Diameter of Fastener (in.)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots[a]	
≤7/8	1/16	1/8	3/4*d*	0
1	1/8	1/8		
≥1 1/8	1/8	3/16		

[a] When length of slot is less than maximum allowable (see Table J3.3), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

TABLE J3.5M
Values of Edge Distance Increment C_2, mm

Nominal Diameter of Fastener (mm)	Oversized Holes	Slotted Holes		
		Long Axis Perpendicular to Edge		Long Axis Parallel to Edge
		Short Slots	Long Slots[a]	
≤22	2	3	0.75*d*	0
24	3	3		
≥27	3	5		

[a] When length of slot is less than maximum allowable (see Table J3.3M), C_2 is permitted to be reduced by one-half the difference between the maximum and actual slot lengths.

(a) For painted members or unpainted members not subject to corrosion, the spacing shall not exceed 24 times the thickness of the thinner plate or 12 in. (305 mm).

(b) For unpainted members of *weathering steel* subject to atmospheric corrosion, the spacing shall not exceed 14 times the thickness of the thinner plate or 7 in. (180 mm).

6. Tension and Shear Strength of Bolts and Threaded Parts

The *design tension* or *shear strength*, ϕR_n, and the *allowable tension* or *shear strength*, R_n/Ω, of a snug-tightened or pretensioned high-strength bolt or threaded part shall be determined according to the *limit states* of *tensile rupture* and *shear rupture* as follows:

$$R_n = F_n A_b \qquad \text{(J3-1)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

F_n = nominal tensile stress F_{nt}, or shear stress, F_{nv} from Table J3.2, ksi (MPa)

A_b = nominal unthreaded body area of bolt or threaded part (for upset rods, see footnote d, Table J3.2), in.^{2} (mm^2)

The required *tensile strength* shall include any tension resulting from *prying action* produced by deformation of the connected parts.

7. Combined Tension and Shear in Bearing-Type Connections

The *available tensile strength* of a bolt subjected to combined tension and shear shall be determined according to the *limit states* of *tension* and *shear rupture* as follows:

$$R_n = F'_{nt} A_b \tag{J3-2}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

where

F'_{nt} = nominal tensile stress modified to include the effects of shearing *stress*, ksi (MPa)

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nt} \text{ (LRFD)} \tag{J3-3a}$$

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt} \text{ (ASD)} \tag{J3-3b}$$

F_{nt} = nominal tensile stress from Table J3.2, ksi (MPa)
F_{nv} = nominal shear stress from Table J3.2, ksi (MPa)
f_v = the required shear stress, ksi (MPa)

The available shear stress of the *fastener* shall equal or exceed the required shear strength per unit area, f_v.

User Note: Note that when the required *stress*, f, in either shear or tension, is less than or equal to 20 percent of the corresponding available stress, the effects of combined *stress* need not be investigated. Also note that Equations J3-3a and J3-3b can be rewritten so as to find a nominal shear stress, F'_{nv}, as a function of the required tensile *stress*, f_t.

8. High-Strength Bolts in Slip-Critical Connections

High-strength bolts in *slip-critical connections* are permitted to be designed to prevent *slip* either as a serviceability limit state or at the required strength limit state. The connection must also be checked for shear strength in accordance with Sections J3.6 and J3.7 and bearing strength in accordance with Sections J3.1 and J3.10.

Slip-critical connections shall be designed as follows, unless otherwise designated by the *engineer of record*. Connections with standard holes or slots transverse to the direction of the load shall be designed for slip as a serviceability limit state. Connections with oversized holes or slots parallel to the direction of the load shall be designed to prevent slip at the required strength level.

The design slip resistance, ϕR_n, and the allowable slip resistance, R_n/Ω, shall be determined for the *limit state* of slip as follows:

$$R_n = \mu D_u h_{sc} T_b N_s \tag{J3-4}$$

For connections in which prevention of slip is a serviceability limit state

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

For connections designed to prevent slip at the *required strength* level

$$\phi = 0.85 \text{ (LRFD)} \qquad \Omega = 1.76 \text{ (ASD)}$$

where

μ = mean slip coefficient for Class A or B surfaces, as applicable, or as established by tests
= 0.35 for Class A surfaces (unpainted clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel and hot-dipped galvanized and roughened surfaces)
= 0.50 for Class B surfaces (unpainted blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

D_u = 1.13; a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension. The use of other values may be approved by the *engineer of record*.

h_{sc} = hole factor determined as follows:

(a) For standard size holes	$h_{sc} = 1.00$
(b) For oversized and short-slotted holes	$h_{sc} = 0.85$
(c) For long-slotted holes	$h_{sc} = 0.70$

N_s = number of slip planes

T_b = minimum fastener tension given in Table J3.1, kips, or J3.1M, kN

User Note: There are special cases where, with oversize holes and slots parallel to the load, the movement possible due to connection slip could cause a structural failure. Resistance and safety factors are provided for connections where slip is prevented until the required strength load is reached.

Design loads are used for either design method and all connections must be checked for strength as bearing-type connections.

9. Combined Tension and Shear in Slip-Critical Connections

When a *slip-critical connection* is subjected to an applied tension that reduces the net clamping *force*, the available *slip* resistance per bolt, from Section J3.8, shall be multiplied by the factor, k_s, as follows:

$$k_s = 1 - \frac{T_u}{D_u T_b N_b} \qquad \text{(LRFD)} \qquad \text{(J3-5a)}$$

$$k_s = 1 - \frac{1.5T_a}{D_u T_b N_b} \qquad \text{(ASD)} \qquad \text{(J3-5b)}$$

where

N_b = number of bolts carrying the applied tension

T_a = tension force due to *ASD load combinations*, kips (kN)

T_b = minimum *fastener* tension given in Table J3.1 or J3.1M, kips (kN)

T_u = tension *force* due to *LRFD load combinations*, kips (kN)

10. Bearing Strength at Bolt Holes

The *available bearing strength*, ϕR_n and R_n/Ω, at bolt holes shall be determined for the *limit state* of *bearing* as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

(a) For a bolt in a *connection* with standard, oversized, and short-slotted holes, independent of the direction of loading, or a long-slotted hole with the slot parallel to the direction of the bearing *force*:

(i) When deformation at the bolt hole at *service load* is a design consideration

$$R_n = 1.2\, L_c t F_u \leq 2.4\, d t F_u \tag{J3-6a}$$

(ii) When deformation at the bolt hole at service load is not a design consideration

$$R_n = 1.5\, L_c t F_u \leq 3.0\, d t F_u \tag{J3-6b}$$

(b) For a bolt in a connection with long-slotted holes with the slot perpendicular to the direction of force:

$$R_n = 1.0\, L_c t F_u \leq 2.0\, d t F_u \tag{J3-6c}$$

(c) For connections made using bolts that pass completely through an unstiffened box member or *HSS*, see Section J7 and Equation J7-1,

where

d = nominal bolt diameter, in. (mm)

F_u = *specified minimum tensile strength* of the connected material, ksi (MPa)

L_c = clear distance, in the direction of the force, between the edge of the hole and the edge of the adjacent hole or edge of the material, in. (mm)

t = thickness of connected material, in. (mm)

For connections, the bearing resistance shall be taken as the sum of the bearing resistances of the individual bolts.

Bearing strength shall be checked for both bearing-type and *slip-critical connections*. The use of oversized holes and short- and long-slotted holes parallel to the line of force is restricted to slip-critical connections per Section J3.2.

11. Special Fasteners

The *nominal strength* of special fasteners other than the bolts presented in Table J3.2 shall be verified by tests.

12. Tension Fasteners

When bolts or other fasteners in tension are attached to an unstiffened box or *HSS* wall, the strength of the wall shall be determined by rational analysis.

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

This section applies to elements of members at *connections* and connecting elements, such as plates, gussets, angles, and brackets.

1. Strength of Elements in Tension

The *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, of affected and connecting elements loaded in tension shall be the lower value obtained according to the *limit states* of *tensile yielding* and *tensile rupture*.

(a) For tensile yielding of connecting elements:

$$R_n = F_y A_g \tag{J4-1}$$

$$\phi = 0.90\,(\text{LRFD}) \qquad \Omega = 1.67\,(\text{ASD})$$

(b) For tensile rupture of connecting elements:

$$R_n = F_u A_e \tag{J4-2}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

where

A_e = *effective net area* as defined in Section D3.3, in.^{2} (mm^2); for bolted *splice* plates, $A_e = A_n \leq 0.85A_g$

2. Strength of Elements in Shear

The available shear yield strength of affected and connecting elements in shear shall be the lower value obtained according to the *limit states* of *shear yielding* and *shear rupture*:

(a) For shear yielding of the element:

$$R_n = 0.60 F_y A_g \tag{J4-3}$$

$$\phi = 1.00\,(\text{LRFD}) \qquad \Omega = 1.50\,(\text{ASD})$$

(b) For shear rupture of the element:

$$R_n = 0.6 F_u A_{nv} \tag{J4-4}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

where

A_{nv} = net area subject to shear, in.^{2} (mm^2)

3. Block Shear Strength

The *available strength* for the *limit state* of *block shear rupture* along a shear failure path or path(s) and a perpendicular tension failure path shall be taken as

$$R_n = 0.6F_u A_{nv} + U_{bs} F_u A_{nt} \leq 0.6 F_y A_{gv} + U_{bs} F_u A_{nt} \tag{J4-5}$$

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

where

A_{gv} = gross area subject to shear, in.^{2} (mm^2)
A_{nt} = *net area* subject to tension, in.^{2} (mm^2)
A_{nv} = net area subject to shear, in.^{2} (mm^2)

Where the tension *stress* is uniform, $U_{bs} = 1$; where the tension stress is nonuniform, $U_{bs} = 0.5$.

User Note: The cases where U_{bs} must be taken equal to 0.5 are illustrated in the Commentary.

4. Strength of Elements in Compression

The available strength of connecting elements in compression for the *limit states* of *yielding* and *buckling* shall be determined as follows.

For $KL/r \leq 25$

$$P_n = F_y A_g \tag{J4-6}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

For $KL/r > 25$ the provisions of Chapter E apply.

J5. FILLERS

In welded construction, any *filler* 1/4 in. (6 mm) or more in thickness shall extend beyond the edges of the *splice* plate and shall be welded to the part on which it is fitted with sufficient weld to transmit the splice plate *load*, applied at the surface of the filler. The welds joining the splice plate to the filler shall be sufficient to transmit the splice plate load and shall be long enough to avoid overloading the filler along the toe of the weld. Any filler less than 1/4 in. (6 mm) thick shall have its edges made flush with the edges of the splice plate and the weld size shall be the sum of the size necessary to carry the splice plate plus the thickness of the filler plate.

When a bolt that carries load passes through fillers that are equal to or less than 1/4 in. (6 mm) thick, the shear strength shall be used without reduction. When a bolt that carries load passes through fillers that are greater than 1/4 in. (6 mm) thick, one of the following requirements shall apply:

(1) For fillers that are equal to or less than 3/4 in. (19 mm) thick, the shear strength of the bolts shall be multiplied by the factor $[1 - 0.4(t - 0.25)]$ [S.I.: $[1 - 0.0154(t - 6)]$], where t is the total thickness of the fillers up to 3/4 in. (19 mm);

(2) The fillers shall be extended beyond the *joint* and the filler extension shall be secured with enough bolts to uniformly distribute the total *force* in the connected element over the combined cross section of the connected element and the fillers;

(3) The size of the joint shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

(4) The joint shall be designed to prevent *slip* at required strength levels in accordance with Section J3.8.

J6. SPLICES

Groove-welded *splices* in *plate girders* and *beams* shall develop the *nominal strength* of the smaller spliced section. Other types of splices in cross sections of plate girders and beams shall develop the strength required by the forces at the point of the splice.

J7. BEARING STRENGTH

The *design bearing strength*, ϕR_n, and the *allowable bearing strength*, R_n/Ω, of surfaces in contact shall be determined for the *limit state* of *bearing (local compressive yielding)* as follows:

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

The nominal bearing strength, R_n, is defined as follows for the various types of bearing:

(a) For *milled surfaces*, pins in reamed, drilled, or bored holes, and ends of *fitted bearing stiffeners*:

$$R_n = 1.8 F_y A_{pb} \tag{J7-1}$$

where

F_y = *specified minimum yield stress*, ksi (MPa)
A_{pb} = projected bearing area, in.^{2} (mm^2)

(b) For *expansion rollers* and rockers:

(i) If $d \leq 25$ in. (635 mm)

$$R_n = 1.2(F_y - 13)ld/20 \tag{J7-2}$$

$$\text{(SI: } R_n = 1.2(F_y - 90)ld/20\text{)} \tag{J7-2M}$$

(ii) If $d > 25$ in. (635 mm)

$$R_n = 6.0(F_y - 13)l\sqrt{d}/20 \tag{J7-3}$$

$$\text{(SI: } R_n = 30.2(F_y - 90)l\sqrt{d}/20\text{)} \tag{J7-3M}$$

where

d = diameter, in. (mm)
l = length of bearing, in. (mm)

J8. COLUMN BASES AND BEARING ON CONCRETE

Proper provision shall be made to transfer the column loads and moments to the footings and foundations.

In the absence of code regulations, the *design bearing strength*, $\phi_c P_p$, and the *allowable bearing strength*, P_p/Ω_c, for the *limit state* of *concrete crushing* are permitted to be taken as follows:

$$\phi_c = 0.60 \text{ (LRFD)} \qquad \Omega_c = 2.50 \text{ (ASD)}$$

The nominal bearing strength, P_p, is determined as follows:

(a) On the full area of a concrete support:

$$P_p = 0.85 f_c' A_1 \tag{J8-1}$$

(b) On less than the full area of a concrete support:

$$P_p = 0.85 f_c' A_1 \sqrt{A_2/A_1} \leq 1.7 f_c' A_1 \tag{J8-2}$$

where

A_1 = area of steel concentrically bearing on a concrete support, in.^{2} (mm^2)
A_2 = maximum area of the portion of the supporting surface that is geometrically similar to and concentric with the loaded area, in.^{2} (mm^2)

J9. ANCHOR RODS AND EMBEDMENTS

Anchor rods shall be designed to provide the required resistance to loads on the completed structure at the base of columns including the net tensile components of any bending moment that may result from load combinations stipulated in Section B2. The anchor rods shall be designed in accordance with the requirements for threaded parts in Table J3.2.

Larger oversized and slotted holes are permitted in base plates when adequate bearing is provided for the nut by using structural or plate washers to bridge the hole.

User Note: The permitted hole sizes and corresponding washer dimensions are given in the AISC *Manual of Steel Construction.*

When horizontal forces are present at *column* bases, these forces should, where possible, be resisted by bearing against concrete elements or by shear friction between the column base plate and the foundation. When anchor rods are designed to resist horizontal force the base plate hole size, the anchor rod setting tolerance, and the horizontal movement of the column shall be considered in the design.

User Note: See ACI 318 for embedment design and for shear friction design. See OSHA for special erection requirements for anchor rods.

J10. FLANGES AND WEBS WITH CONCENTRATED FORCES

This section applies to *single-* and *double-concentrated forces* applied normal to the flange(s) of wide flange sections and similar built-up shapes. A single-concentrated force can be either tensile or compressive. Double-concentrated forces are one tensile and one compressive and form a couple on the same side of the loaded member.

When the *required strength* exceeds the *available strength* as determined for the *limit states* listed in this section, *stiffeners* and/or *doublers* shall be provided and shall be sized for the difference between the required strength and the available strength for the applicable *limit state*. Stiffeners shall also meet the design requirements in Section J10.8. Doublers shall also meet the design requirement in Section J10.9

User Note: See Appendix 6.3 for requirements for the ends of cantilever members.

Stiffeners are required at *unframed ends* of *beams* in accordance with the requirements of Section J10.7.

1. Flange Local Bending

This section applies to tensile *single-concentrated forces* and the tensile component of *double-concentrated forces*.

The *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, for the *limit state* of flange *local bending* shall be determined as follows:

$$R_n = 6.25 t_f^{\,2} F_{yf} \qquad \text{(J10-1)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where

F_{yf} = *specified minimum yield stress* of the flange, ksi (MPa)
t_f = thickness of the loaded flange, in. (mm)

If the length of loading across the member flange is less than $0.15b_f$, where b_f is the member flange width, Equation J10-1 need not be checked.

When the concentrated *force* to be resisted is applied at a distance from the member end that is less than $10t_f$, R_n shall be reduced by 50 percent.

When required, a pair of *transverse stiffeners* shall be provided.

2. Web Local Yielding

This section applies to *single-concentrated forces* and both components of *double-concentrated forces*.

The *available strength* for the *limit state* of web *local yielding* shall be determined as follows:

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

The *nominal strength*, R_n, shall be determined as follows:

(a) When the concentrated *force* to be resisted is applied at a distance from the member end that is greater than the depth of the member d,

$$R_n = (5k + N) F_{yw} t_w \qquad \text{(J10-2)}$$

(b) When the concentrated force to be resisted is applied at a distance from the member end that is less than or equal to the depth of the member d,

$$R_n = (2.5k + N)F_{yw}t_w \tag{J10-3}$$

where

k = distance from outer face of the flange to the web toe of the fillet, in. (mm)
F_{yw} = *specified minimum yield stress* of the web, ksi (MPa)
N = length of bearing (not less than k for end *beam* reactions), in. (mm)
t_w = web thickness, in. (mm)

When required, a pair of *transverse stiffeners* or a *doubler* plate shall be provided.

3. Web Crippling

This section applies to compressive *single-concentrated forces* or the compressive component of *double-concentrated forces*.

The *available strength* for the *limit state* of *web local crippling* shall be determined as follows:

$$\phi = 0.75\,(\text{LRFD}) \qquad \Omega = 2.00\,(\text{ASD})$$

The *nominal strength*, R_n, shall be determined as follows:

(a) When the concentrated compressive *force* to be resisted is applied at a distance from the member end that is greater than or equal to $d/2$:

$$R_n = 0.80t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{J10-4}$$

(b) When the concentrated compressive force to be resisted is applied at a distance from the member end that is less than $d/2$:

(i) For $N/d \le 0.2$

$$R_n = 0.40t_w^2\left[1 + 3\left(\frac{N}{d}\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{J10-5a}$$

(ii) For $N/d > 0.2$

$$R_n = 0.40t_w^2\left[1 + \left(\frac{4N}{d} - 0.2\right)\left(\frac{t_w}{t_f}\right)^{1.5}\right]\sqrt{\frac{EF_{yw}t_f}{t_w}} \tag{J10-5b}$$

where

d = overall depth of the member, in. (mm)
t_f = flange thickness, in. (mm)

When required, a *transverse stiffener*, or pair of transverse stiffeners, or a *doubler* plate extending at least one-half the depth of the web shall be provided.

4. Web Sidesway Buckling

This Section applies only to compressive *single-concentrated forces* applied to members where relative lateral movement between the loaded compression flange and the tension flange is not restrained at the point of application of the concentrated *force*.

The *available strength* of the web shall be determined as follows:

$$\phi = 0.85 \text{ (LRFD)} \qquad \Omega = 1.76 \text{ (ASD)}$$

The *nominal strength*, R_n, for the *limit state* of *web sidesway buckling* shall be determined as follows:

(a) If the compression flange is restrained against rotation:

(i) For $(h/t_w)/(l/b_f) \leq 2.3$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[1 + 0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \qquad \text{(J10-6)}$$

(ii) For $(h/t_w)/(l/b_f) > 2.3$, the limit state of web sidesway buckling does not apply.

When the *required strength* of the web exceeds the available strength, local *lateral bracing* shall be provided at the tension flange or either a pair of *transverse stiffeners* or a *doubler* plate shall be provided.

(b) If the compression flange is not restrained against rotation:

(i) For $(h/t_w)/(l/b_f) \leq 1.7$

$$R_n = \frac{C_r t_w^3 t_f}{h^2}\left[0.4\left(\frac{h/t_w}{l/b_f}\right)^3\right] \qquad \text{(J10-7)}$$

(ii) For $(h/t_w)/(l/b_f) > 1.7$, the limit state of web sidesway buckling does not apply.

When the required strength of the web exceeds the available strength, local *lateral bracing* shall be provided at both flanges at the point of application of the concentrated forces.

In Equations J10-6 and J10-7, the following definitions apply:

b_f = flange width, in. (mm)
C_r = 960,000 ksi (6.62×10^6 MPa) when $M_u < M_y$ (LRFD) or $1.5M_a < M_y$ (ASD) at the location of the force
= 480,000 ksi (3.31×10^6 MPa) when $M_u \geq M_y$ (LRFD) or $1.5M_a \geq M_y$ (ASD) at the location of the force
h = clear distance between flanges less the fillet or corner radius for rolled shapes; distance between adjacent lines of *fasteners* or the clear distance between flanges when welds are used for built-up shapes, in. (mm)
l = largest laterally *unbraced length* along either flange at the point of *load*, in. (mm)
t_f = flange thickness, in. (mm)
t_w = web thickness, in. (mm)

User Note: For determination of adequate restraint, refer to Appendix 6.

5. Web Compression Buckling

This Section applies to a pair of compressive *single-concentrated forces* or the compressive components in a pair of *double-concentrated forces*, applied at both flanges of a member at the same location.

The *available strength* for the *limit state* of web *local buckling* shall be determined as follows:

$$R_n = \frac{24t_w^3\sqrt{EF_{yw}}}{h} \tag{J10-8}$$

$$\phi = 0.90\,(\text{LRFD}) \qquad \Omega = 1.67\,(\text{ASD})$$

When the pair of concentrated compressive *forces* to be resisted is applied at a distance from the member end that is less than $d/2$, R_n shall be reduced by 50 percent.

When required, a single *transverse stiffener*, a pair of transverse stiffeners, or a *doubler* plate extending the full depth of the web shall be provided.

6. Web Panel Zone Shear

This section applies to *double-concentrated forces* applied to one or both flanges of a member at the same location.

The *available strength* of the web *panel zone* for the *limit state* of *shear yielding* shall be determined as follows:

$$\phi = 0.90\ (\text{LRFD}) \qquad \Omega = 1.67\ (\text{ASD})$$

The *nominal strength*, R_n, shall be determined as follows:

(a) When the effect of panel-zone deformation on frame *stability* is not considered in the analysis:

(i) For $P_r \leq 0.4P_c$

$$R_n = 0.60F_y d_c t_w \tag{J10-9}$$

(ii) For $P_r > 0.4P_c$

$$R_n = 0.60F_y d_c t_w \left(1.4 - \frac{P_r}{P_c}\right) \tag{J10-10}$$

(b) When frame stability, including plastic panel-zone deformation, is considered in the analysis:

(i) For $P_r \leq 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right) \tag{J10-11}$$

(ii) For $P_r > 0.75P_c$

$$R_n = 0.60F_y d_c t_w \left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)\left(1.9 - \frac{1.2P_r}{P_c}\right) \tag{J10-12}$$

In Equations J10-9 through J10-12, the following definitions apply:

A = *column* cross-sectional area, in.^{2} (mm^2)
b_{cf} = width of column flange, in. (mm)
d_b = *beam* depth, in. (mm)
d_c = column depth, in. (mm)
F_y = *specified minimum yield stress* of the column web, ksi (MPa)
P_c = P_y, kips (N) (LRFD)
P_c = $0.6P_y$, kips (N) (ASD)
P_r = *required strength*, kips (N)
P_y = F_yA, axial *yield strength* of the column, kips (N)
t_{cf} = thickness of the column flange, in. (mm)
t_w = column web thickness, in. (mm)

When required, *doubler* plate(s) or a pair of *diagonal stiffeners* shall be provided within the boundaries of the rigid connection whose webs lie in a common plane.

See Section J10.9 for doubler plate design requirements.

7. Unframed Ends of Beams and Girders

At *unframed ends* of *beams* and *girders* not otherwise restrained against rotation about their longitudinal axes, a pair of *transverse stiffeners*, extending the full depth of the web, shall be provided.

8. Additional Stiffener Requirements for Concentrated Forces

Stiffeners required to resist tensile concentrated *forces* shall be designed in accordance with the requirements of Chapter D and welded to the loaded flange and the web. The welds to the flange shall be sized for the difference between the *required strength* and available *limit state* strength. The stiffener to web welds shall be sized to transfer to the web the algebraic difference in tensile force at the ends of the stiffener.

Stiffeners required to resist compressive concentrated forces shall be designed in accordance with the requirements in Sections E6.2 and J4.4 and shall either bear on or be welded to the loaded flange and welded to the web. The welds to the flange shall be sized for the difference between the required strength and the applicable limit state strength. The weld to the web shall be sized to transfer to the web the algebraic difference in compression force at the ends of the stiffener. For fitted bearing stiffeners, see Section J7.

Transverse full depth bearing stiffeners for compressive forces applied to a *beam* or *plate girder* flange(s) shall be designed as axially compressed members (*columns*) in accordance with the requirements of Sections E6.2 and J4.4.

The member properties shall be determined using an *effective length* of $0.75h$ and a cross section composed of two stiffeners and a strip of the web having a width of $25t_w$ at interior stiffeners and $12t_w$ at the ends of members. The weld connecting full depth bearing stiffeners to the web shall be sized to transmit the difference in compressive force at each of the stiffeners to the web.

Transverse and *diagonal stiffeners* shall comply with the following additional criteria:

(1) The width of each *stiffener* plus one-half the thickness of the column web shall not be less than one-third of the width of the flange or moment connection plate delivering the concentrated force.
(2) The thickness of a stiffener shall not be less than one-half the thickness of the flange or moment connection plate delivering the concentrated *load*, and greater than or equal to the width divided by 15.
(3) *Transverse stiffeners* shall extend a minimum of one-half the depth of the member except as required in J10.5 and J10.7.

9. Additional Doubler Plate Requirements for Concentrated Forces

Doubler plates required for compression strength shall be designed in accordance with the requirements of Chapter E.

Doubler plates required for tensile strength shall be designed in accordance with the requirements of Chapter D.

Doubler plates required for shear strength (see Section J10.6) shall be designed in accordance with the provisions of Chapter G.

In addition, doubler plates shall comply with the following criteria:
(1) The thickness and extent of the doubler plate shall provide the additional material necessary to equal or exceed the strength requirements.
(2) The doubler plate shall be welded to develop the proportion of the total force transmitted to the doubler plate.

CHAPTER K

DESIGN OF HSS AND BOX MEMBER CONNECTIONS

This chapter covers member strength design considerations pertaining to connections to HSS members and box sections of uniform wall thickness. See also Chapter J for additional requirements for bolting to HSS.

The chapter is organized as follows:

K1. Concentrated Forces on HSS
K2. HSS-to-HSS Truss Connections
K3. HSS-to-HSS Moment Connections

User Note: See Section J3.10(c) for through-bolts.

K1. CONCENTRATED FORCES ON HSS

1. Definitions of Parameters

B = overall width of rectangular *HSS* member, measured 90 degrees to the plane of the *connection*, in. (mm)
B_p = width of plate, measured 90 degrees to the plane of the connection, in. (mm)
D = outside diameter of round HSS member, in. (mm)
F_y = *specified minimum yield stress* of HSS member material, ksi (MPa)
F_{yp} = specified minimum yield stress of plate, ksi (MPa)
F_u = *specified minimum tensile strength* of HSS material, ksi (MPa)
H = overall height of rectangular HSS member, measured in the plane of the connection, in. (mm)
N = bearing length of the *load*, measured parallel to the axis of the HSS member, (or measured across the width of the HSS in the case of loaded cap plates), in. (mm)
t = *design wall thickness* of HSS member, in. (mm)
t_p = thickness of plate, in. (mm)

2. Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits of applicability:

(1) Strength: $F_y \leq 52$ ksi (360 MPa) for *HSS*
(2) Ductility: $F_y/F_u \leq 0.8$ for HSS
(3) Other limits apply for specific criteria

3. Concentrated Force Distributed Transversely

3a. Criterion for Round HSS

When a concentrated *force* is distributed transversely to the axis of the *HSS* the *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, for the *limit state* of *local yielding* shall be determined as follows:

$$R_n = F_y t^2[5.5/(1 - 0.81 B_p/D)]Q_f \quad \text{(K1-1)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation K2-1.

Additional limits of applicability are

(1) $0.2 < B_p/D \le 1.0$
(2) $D/t \le 50$ for *T-connections* and $D/t \le 40$ for *cross-connections*

3b. Criteria for Rectangular HSS

When a concentrated force is distributed transversely to the axis of the HSS the design strength, ϕR_n, and the allowable strength, R_n/Ω, shall be the lowest value according to the limit states of local yielding due to *uneven load distribution*, *shear yielding (punching)* and sidewall strength.

Additional limits of applicability are

(1) $0.25 < B_p/B \le 1.0$
(2) B/t for the loaded HSS wall ≤ 35

(a) For the limit state of local yielding due to uneven load distribution in the loaded plate,

$$R_n = [10 F_y t/(B/t)]B_p \le F_{yp} t_p B_p \quad \text{(K1-2)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$R_n = 0.6 F_y t\ [2t_p + 2B_{ep}] \quad \text{(K1-3)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

$$B_{ep} = 10 B_p/(B/t) \le B_p$$

This limit state need not be checked when $B_p > (B - 2t)$, nor when $B_p < 0.85B$.

(c) For the limit state of sidewall under tension loading, the available strength shall be taken as the strength for sidewall local yielding. For the limit state of sidewall under compression loading, available strength shall be taken as the lowest value obtained according to the limit states of sidewall local yielding, sidewall local crippling and sidewall local buckling.

This limit state need not be checked unless the *chord member* and *branch member* (connecting element) have the same width ($\beta = 1.0$).

(i) For the limit state of sidewall local yielding,

$$R_n = 2F_y t[5k + N] \tag{K1-4}$$

$$\phi = 1.0 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

k = outside corner radius of the HSS, which is permitted to be taken as $1.5t$ if unknown, in. (mm)

(ii) For the limit state of sidewall local crippling, in T-connections,

$$R_n = 1.6t^2[1 + 3N/(H - 3t)]\,(EF_y)^{0.5} Q_f \tag{K1-5}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.0 \text{ (ASD)}$$

where Q_f is given by Equation K2-10.

(iii) For the limit state of sidewall local buckling in cross-connections,

$$R_n = [48t^3/(H - 3t)]\,(EF_y)^{0.5} Q_f \tag{K1-6}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation K2-10.

The nonuniformity of load transfer along the line of weld, due to the flexibility of the HSS wall in a transverse plate-to-HSS connection, shall be considered in proportioning such welds. This requirement can be satisfied by limiting the total effective weld length, L_e, of groove and fillet welds to rectangular HSS as follows:

$$L_e = 2[10/(B/t)]\,[(F_y t)/(F_{yp} t_p)] B_p \leq 2B_p \tag{K1-7}$$

where

L_e = total effective weld length for welds on both sides of the transverse plate, in. (mm)

In lieu of Equation K1-7, this requirement may be satisfied by other rational approaches.

> **User Note**: An upper limit on weld size will be given by the weld that develops the available strength of the connected element.

4. Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width, and Acting Perpendicular to the HSS Axis

When a concentrated *force* is distributed longitudinally along the axis of the *HSS* at the center of the HSS diameter or width, and also acts perpendicular to the axis direction of the HSS (or has a component perpendicular to the axis direction of the

HSS), the *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, perpendicular to the HSS axis shall be determined for the *limit state* of *chord plastification* as follows.

4a. Criterion for Round HSS

An additional limit of applicability is:
$D/t \leq 50$ for *T-connections* and $D/t \leq 40$ for *cross-connections*

$$R_n = 5.5 F_y t^2 (1 + 0.25 N/D) Q_f \tag{K1-8}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

where Q_f is given by Equation K2-1.

4b. Criterion for Rectangular HSS

An additional limit of applicability is:
B/t for the loaded HSS wall ≤ 40

$$R_n = [F_y t^2/(1 - t_p/B)]\,[2N/B + 4(1 - t_p/B)^{0.5} Q_f] \tag{K1-9}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

$Q_f = (1 - U^2)^{0.5}$
U is given by Equation K2-12

5. Concentrated Force Distributed Longitudinally at the Center of the HSS Width, and Acting Parallel to the HSS Axis

When a concentrated *force* is distributed longitudinally along the axis of a rectangular *HSS*, and also acts parallel but eccentric to the axis direction of the member, the *connection* shall be verified as follows:

$$F_{yp} t_p \leq F_u t \tag{K1-10}$$

User Note: This provision is primarily intended for shear tab connections. Equation K1-10 precludes shear yielding (punching) of the HSS wall by requiring the plate (shear tab) strength to be less than the HSS wall strength. For bracing connections to HSS columns, where a load is applied by a longitudinal plate at an angle to the HSS axis, the connection design will be governed by the force component perpendicular to the HSS axis (see Section K1.4b).

6. Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate

When a concentrated *force* acts on the end of a capped *HSS*, and the force is in the direction of the HSS axis, the *design strength*, ϕR_n, and the *allowable strength*, R_n/Ω, shall be determined for the *limit states* of wall *local yielding* (due to tensile or compressive *forces*) and wall *local crippling* (due to compressive forces only), with consideration for shear lag, as follows.

User Note: The procedure below presumes that the concentrated *force* has a dispersion slope of 2.5:1 through the cap plate (of thickness t_p) and disperses into the two HSS walls of dimension B.

If $(5t_p + N) \geq B$, the *available strength* of the HSS is computed by summing the contributions of all four HSS walls.

If $(5t_p + N) < B$, the available strength of the HSS is computed by summing the contributions of the two walls into which the *load* is distributed.

(i) For the limit state of wall local yielding, for one wall,

$$R_n = F_y t[5t_p + N] \leq BF_y t \qquad \text{(K1-11)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

(ii) For the limit state of wall local crippling, for one wall,

$$R_n = 0.8t^2[1 + (6N/B)(t/t_p)^{1.5}][EF_y t_p/t]^{0.5} \qquad \text{(K1-12)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

K2. HSS-TO-HSS TRUSS CONNECTIONS

HSS-to-HSS truss *connections* are defined as connections that consist of one or more *branch members* that are directly welded to a continuous chord that passes through the connection and shall be classified as follows:

(a) When the *punching load* ($P_r \sin\theta$) in a branch member is equilibrated by *beam* shear in the *chord member*, the connection shall be classified as a *T-connection* when the branch is perpendicular to the chord and a *Y-connection* otherwise.

(b) When the punching load ($P_r \sin\theta$) in a branch member is essentially equilibrated (within 20 percent) by *loads* in other branch member(s) on the same side of the connection, the connection shall be classified as a *K-connection*. The relevant gap is between the primary branch members whose loads equilibrate. An *N-connection* can be considered as a type of K-connection.

User Note: A K-connection with one branch perpendicular to the chord is often called an N-connection.

(c) When the punching load ($P_r \sin\theta$) is transmitted through the chord member and is equilibrated by branch member(s) on the opposite side, the connection shall be classified as a *cross-connection*.

(d) When a connection has more than two primary branch members, or branch members in more than one plane, the connection shall be classified as a general or multiplanar connection.

When branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the *nominal strength* shall be determined by interpolation on the proportion of each in total.

For the purposes of this Specification, the centerlines of branch members and chord members shall lie in a common plane. Rectangular HSS connections are further limited to have all members oriented with walls parallel to the plane. For trusses that are made with HSS that are connected by welding branch members to chord members, eccentricities within the limits of applicability are permitted without consideration of the resulting moments for the design of the connection.

1. Definitions of Parameters

B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, in. (mm)

B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, in. (mm)

D = outside diameter of round HSS main member, in. (mm)

D_b = outside diameter of round HSS branch member, in. (mm)

e = eccentricity in a truss connection, positive being away from the branches, in. (mm)

F_y = *specified minimum yield stress* of HSS main member material, ksi (MPa)

F_{yb} = specified minimum yield stress of HSS branch member material, ksi (MPa)

F_u = *specified minimum tensile strength* of HSS material, ksi (MPa)

g = gap between toes of branch members in a gapped K-connection, neglecting the welds, in. (mm)

H = overall height of rectangular HSS main member, measured in the plane of the connection, in. (mm)

H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)

t = *design wall thickness* of HSS main member, in. (mm)

t_b = design wall thickness of HSS branch member, in. (mm)

β = the width ratio; the ratio of branch diameter to chord diameter $= D_b/D$ for round HSS; the ratio of overall branch width to chord width $= B_b/B$ for rectangular HSS

β_{eff} = the *effective width* ratio; the sum of the perimeters of the two branch members in a K-connection divided by eight times the chord width

γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness $= D/2t$ for round HSS; the ratio of one-half the width to wall thickness $= B/2t$ for rectangular HSS

η = the *load* length parameter, applicable only to rectangular *HSS*; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width $= N/B$, where $N = H_b/\sin\theta$

θ = acute angle between the branch and chord (degrees)

ζ = the gap ratio; the ratio of the gap between the branches of a gapped K-connection to the width of the chord $= g/B$ for rectangular HSS

2. Criteria for Round HSS

The interaction of *stress* due to *chord member forces* and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \tag{K2-1}$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \tag{K2-2}$$

and

P_r = required axial strength in chord, kips (N); for K-connections, P_r is to be determined on the side of the *joint* that has the lower compression stress (lower U)
M_r = required flexural strength in chord, kip-in. (N-mm)
A_g = chord gross area, in.^{2} (mm^2)
F_c = *available stress*, ksi (MPa)
S = chord elastic section modulus, in.^{3} (mm^3)

For design according to Section B3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, kips (N)
$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)
$F_c = F_y$, ksi (MPa)

For design according to Section B3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, kips (N)
$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)
$F_c = 0.6\ F_y$, ksi (MPa)

2a. Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits of applicability:

(1) *Joint eccentricity*: $-0.55D \leq e \leq 0.25D$, where D is the chord diameter and e is positive away from the branches
(2) Branch angle: $\theta \geq 30°$
(3) Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for T-, Y- and *K-connections*; less than or equal to 40 for *cross-connections*

(4) Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50

(5) Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05E/F_y$

(6) Width ratio: $0.2 < D_b/D \leq 1.0$ in general, and $0.4 \leq D_b/D \leq 1.0$ for gapped K-connections

(7) If a *gap connection*: g greater than or equal to the sum of the branch wall thicknesses

(8) If an *overlap connection*: $25\% \leq O_v \leq 100\%$, where $O_v = (q/p) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal diameter, the thicker) branch is a "thru member" connected directly to the chord.

(9) Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch

(10) Strength: $F_y \leq 52$ ksi (360 MPa) for chord and branches

(11) Ductility: $F_y\,/\,F_u \leq 0.8$

2b. Branches with Axial Loads in T-, Y- and Cross-Connections

For T- and Y- connections, the *design strength* of the branch ϕP_n, or the *allowable strength* of the branch, P_n/Ω, shall be the lower value obtained according to the *limit states* of *chord plastification* and *shear yielding (punching).*

(a) For the limit state of chord plastification in T- and Y-connections,

$$P_n \sin\theta = F_y t^2[3.1 + 15.6\beta^2]\gamma^{0.2} Q_f \qquad \text{(K2-3)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$P_n = 0.6 F_y t \pi D_b[(1 + \sin\theta)/2\sin^2\theta] \qquad \text{(K2-4)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

(c) For the limit state of chord plastification in cross-connections,

$$P_n \sin\theta = F_y t^2[5.7/(1 - 0.81\beta)]Q_f \qquad \text{(K2-5)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

2c. Branches with Axial Loads in K-Connections

For K-connections, the design strength of the branch, ϕP_n, and the allowable strength of the branch, P_n/Ω, shall be the lower value obtained according to the limit states of chord plastification for gapped and overlapped connections and shear yielding (punching) for gapped connections only.

(a) For the limit state of chord plastification,

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

For the compression branch:

$$P_n \sin\theta = F_y t^2 [2.0 + 11.33 D_b/D] Q_g Q_f \quad \text{(K2-6)}$$

where D_b refers to the compression branch only, and

$$Q_g = \gamma^{0.2} \left[1 + \frac{0.024\gamma^{1.2}}{e^{\left(\frac{0.5g}{t} - 1.33\right)} + 1} \right] \quad \text{(K2-7)}$$

In gapped connections, g (measured along the crown of the chord neglecting weld dimensions) is positive. In overlapped connections, g is negative and equals q.

For the tension branch,

$$P_n \sin\theta = (P_n \sin\theta)_{\text{compression branch}} \quad \text{(K2-8)}$$

(b) For the limit state of shear yielding (punching) in gapped K-connections,

$$P_n = 0.6 F_y t \pi D_b [(1 + \sin\theta)/2\sin^2\theta] \quad \text{(K2-9)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

3. Criteria for Rectangular HSS

The interaction of *stress* due to *chord member forces* and local branch connection forces shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression in T-, Y-, and *cross-connections*,

$$Q_f = 1.3 - 0.4U/\beta \le 1 \quad \text{(K2-10)}$$

When the chord is in compression in gapped *K-connections*,

$$Q_f = 1.3 - 0.4U/\beta_{eff} \le 1 \quad \text{(K2-11)}$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \quad \text{(K2-12)}$$

and

P_r = required axial strength in chord, kips (N). For gapped K-connections, P_r is to be determined on the side of the *joint* that has the higher compression stress (higher U).

M_r = required flexural strength in chord, kip-in. (N-mm)

A_g = chord gross area, in.^{2} (mm^2)

F_c = *available stress*, ksi (MPa)

S = chord elastic section modulus, in.^{3} (mm^3)

For design according to Section B3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, kips (N)
$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)
$F_c = F_y$, ksi (MPa)

For design according to Section B3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, kips, (N)
$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)
$F_c = 0.6F_y$, ksi, (MPa)

3a. Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits:

(1) *Joint eccentricity*: $-0.55H \le e \le 0.25H$, where H is the chord depth and e is positive away from the branches
(2) Branch angle: $\theta \ge 30°$
(3) Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to 30 for overlapped K-connections
(4) Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35
(5) Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35 for gapped K-connections and T-, Y- and cross-connections; less than or equal to $1.1(E/F_{yb})^{0.5}$ for overlapped K-connections
(6) Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25 for T-, Y-, cross- and overlapped K-connections; greater than or equal to 0.35 for gapped K-connections
(7) Aspect ratio: $0.5 \le$ ratio of depth to width ≤ 2.0
(8) Overlap: $25\% \le O_v \le 100\%$, where $O_v = (q/p) \times 100\%$. p is the projected length of the overlapping branch on the chord; q is the overlap length measured along the connecting face of the chord beneath the two branches. For overlap connections, the larger (or if equal width, the thicker) branch is a "thru member" connected directly to the chord
(9) Branch width ratio for *overlap connections*: ratio of overall wall width of overlapping branch to overall wall width of overlapped branch greater than or equal to 0.75
(10) Branch thickness ratio for overlap connections: thickness of overlapping branch to be less than or equal to the thickness of the overlapped branch

(11) Strength: $F_y \leq 52$ ksi (360 MPa) for chord and branches
(12) Ductility: $F_y/F_u \leq 0.8$
(13) Other limits apply for specific criteria

3b. Branches with Axial Loads in T-, Y- and Cross-Connections

For T-, Y-, and cross-connections, the *design strength* of the branch, ϕP_n, or the *allowable strength* of the branch, P_n/Ω, shall be the lowest value obtained according to the *limit states* of *chord wall plastification*, *shear yielding (punching)*, sidewall strength and *local yielding* due to *uneven load distribution*. In addition to the limits of applicability in Section K2.3a, β shall not be less than 0.25.

(a) For the limit state of chord wall plastification,

$$P_n \sin\theta = F_y t^2 [2\eta/(1-\beta) + 4/(1-\beta)^{0.5}] Q_f \tag{K2-13}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked when $\beta > 0.85$.

(b) For the limit state of shear yielding (punching),

$$P_n \sin\theta = 0.6 F_y t B [2\eta + 2\beta_{eop}] \tag{K2-14}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

In Equation K2-14, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β.

This limit state need not be checked when $\beta > (1 - 1/\gamma)$, nor when $\beta < 0.85$ and $B/t \geq 10$.

(c) For the limit state of sidewall strength, the *available strength* for branches in tension shall be taken as the available strength for sidewall local yielding. For the limit state of sidewall strength, the available strength for branches in compression shall be taken as the lower of the strengths for sidewall local yielding and sidewall local crippling. For cross-connections with a branch angle less than 90 degrees, an additional check for chord sidewall shear failure must be made in accordance with Section G5.

This limit state need not be checked unless the chord member and branch member have the same width ($\beta = 1.0$).

(i) For the limit state of local yielding,

$$P_n \sin\theta = 2 F_y t [5k + N] \tag{K2-15}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

k = outside corner radius of the *HSS*, which is permitted to be taken as $1.5t$ if unknown, in. (mm)
N = bearing length of the *load*, parallel to the axis of the HSS main member, $H_b/\sin\theta$, in. (mm)

(ii) For the limit state of sidewall local crippling, in T- and Y-connections,

$$P_n \sin\theta = 1.6t^2[1 + 3N/(H - 3t)](EF_y)^{0.5} Q_f \qquad \text{(K2-16)}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

(iii) For the limit state of sidewall local crippling in cross-connections,

$$P_n \sin\theta = [48t^3/(H - 3t)](EF_y)^{0.5} Q_f \qquad \text{(K2-17)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(d) For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb} t_b [2H_b + 2b_{eoi} - 4t_b] \qquad \text{(K2-18)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

$$b_{eoi} = [10/(B/t)][F_y t/(F_{yb} t_b)]B_b \le B_b \qquad \text{(K2-19)}$$

This limit state need not be checked when $\beta < 0.85$.

3c. Branches with Axial Loads in Gapped K-Connections

For gapped K-connections, the design strength of the branch, ϕP_n, or the allowable strength of the branch, P_n/Ω, shall be the lowest value obtained according to the limit states of chord wall plastification, shear yielding (punching), *shear yielding* and local yielding due to uneven load distribution. In addition to the limits of applicability in Section K2.3a, the following limits shall apply:

(1) $B_b/B \ge 0.1 + \gamma/50$
(2) $\beta_{eff} \ge 0.35$
(3) $\zeta \ge 0.5(1 - \beta_{eff})$
(4) Gap: g greater than or equal to the sum of the branch wall thicknesses
(5) The smaller $B_b > 0.63$ times the larger B_b

(a) For the limit state of chord wall plastification,

$$P_n \sin\theta = F_y t^2 [9.8\beta_{eff}\gamma^{0.5}]Q_f \qquad \text{(K2-20)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$P_n \sin\theta = 0.6F_y t B[2\eta + \beta + \beta_{eop}] \qquad \text{(K2-21)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

In the above equation, the effective outside punching parameter $\beta_{eop} = 5\beta/\gamma$ shall not exceed β.

This limit state need only be checked if $B_b < (B - 2t)$ or the branch is not square.

(c) For the limit state of shear yielding of the chord in the gap, available strength shall be checked in accordance with Section G5. This limit state need only be checked if the chord is not square.

(d) For the limit state of local yielding due to uneven load distribution,

$$P_n = F_{yb}t_b[2H_b + B_b + b_{eoi} - 4t_b] \tag{K2-22}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

$$b_{eoi} = [10/(B/t)][F_y t/(F_{yb}t_b)]B_b \leq B_b \tag{K2-23}$$

This limit state need only be checked if the branch is not square or $B/t < 15$.

3d. Branches with Axial Loads in Overlapped K-Connections

For overlapped K-connections, the design strength of the branch, ϕP_n, or the allowable strength of the branch, P_n/Ω, shall be determined from the limit state of local yielding due to uneven load distribution,

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

For the overlapping branch, and for overlap $25\% \leq O_v \leq 50\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi}t_{bi}[(O_v/50)(2H_{bi} - 4t_{bi}) + b_{eoi} + b_{eov}] \tag{K2-24}$$

For the overlapping branch, and for overlap $50\% \leq O_v < 80\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi}t_{bi}[2H_{bi} - 4t_{bi} + b_{eoi} + b_{eov}] \tag{K2-25}$$

For the overlapping branch, and for overlap $80\% \leq O_v \leq 100\%$ measured with respect to the overlapping branch,

$$P_n = F_{ybi}t_{bi}[2H_{bi} - 4t_{bi} + B_{bi} + b_{eov}] \tag{K2-26}$$

where

b_{eoi} is the *effective width* of the *branch face* welded to the chord,

$$b_{eoi} = [10/(B/t)][(F_y t)/(F_{ybi}t_{bi})]B_{bi} \leq B_{bi} \tag{K2-27}$$

b_{eov} is the effective width of the branch face welded to the overlapped brace,

$$b_{eov} = [10/(B_{bj}/t_{bj})][(F_{ybj}t_{bj})/(F_{ybi}t_{bi})]\, B_{bi} \leq B_{bi} \tag{K2-28}$$

B_{bi} = overall branch width of the overlapping branch, in. (mm)
B_{bj} = overall branch width of the overlapped branch, in. (mm)
F_{ybi} = *specified minimum yield stress* of the overlapping branch material, ksi (MPa)
F_{ybj} = specified minimum yield stress of the overlapped branch material, ksi (MPa)
H_{bi} = overall depth of the overlapping branch, in. (mm)
t_{bi} = thickness of the overlapping branch, in. (mm)
t_{bj} = thickness of the overlapped branch, in. (mm)

For the overlapped branch, P_n shall not exceed P_n of the overlapping branch, calculated using Equation K2-24, K2-25, or K2-26, as applicable, multiplied by the factor $(A_{bj} F_{ybj} / A_{bi} F_{ybi})$,

where

A_{bi} = cross-sectional area of the overlapping branch
A_{bj} = cross-sectional area of the overlapped branch

3e. Welds to Branches

The nonuniformity of load transfer along the line of weld, due to differences in relative flexibility of HSS walls in HSS-to-HSS connections, shall be considered in proportioning such welds. This can be considered by limiting the total effective weld length, L_e, of groove and *fillet welds* to rectangular HSS as follows:

(a) In T-, Y- and cross-connections,

for $\theta \leq 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b) \tag{K2-29}$$

for $\theta \geq 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} \tag{K2-30}$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

(b) In gapped K-connections, around each branch,

for $\theta \leq 50$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + 2(B_b - 1.2t_b) \tag{K2-31}$$

for $\theta \geq 60$ degrees

$$L_e = \frac{2(H_b - 1.2t_b)}{\sin\theta} + (B_b - 1.2t_b) \tag{K2-32}$$

Linear interpolation shall be used to determine L_e for values of θ between 50 and 60 degrees.

In lieu of the above criteria in Equations K2-29 to K2-32, other rational criteria are permitted.

K3. HSS-TO-HSS MOMENT CONNECTIONS

HSS-to-HSS *moment connections* are defined as *connections* that consist of one or two *branch members* that are directly welded to a continuous chord that passes through the connection, with the branch or branches loaded by bending moments. A connection shall be classified

(a) As a *T-connection* when there is one branch and it is perpendicular to the chord and as a *Y-connection* when there is one branch but not perpendicular to the chord.

(b) As a *cross-connection* when there is a branch on each (opposite) side of the chord.

For the purposes of this Specification, the centerlines of the branch member(s) and the *chord member* shall lie in a common plane.

1. Definitions of Parameters

B = overall width of rectangular HSS main member, measured 90 degrees to the plane of the connection, in. (mm)

B_b = overall width of rectangular HSS branch member, measured 90 degrees to the plane of the connection, in. (mm)

D = outside diameter of round HSS main member, in. (mm)

D_b = outside diameter of round HSS branch member, in. (mm)

F_y = *specified minimum yield stress* of HSS main member, ksi (MPa)

F_{yb} = specified minimum yield stress of HSS branch member, ksi (MPa)

F_u = ultimate strength of HSS member, ksi (MPa)

H = overall height of rectangular HSS main member, measured in the plane of the connection, in. (mm)

H_b = overall height of rectangular HSS branch member, measured in the plane of the connection, in. (mm)

t = *design wall thickness* of HSS main member, in. (mm)

t_b = design wall thickness of HSS branch member, in. (mm)

β = the width ratio; the ratio of branch diameter to chord diameter = D_b/D for round HSS; the ratio of overall branch width to chord width = B_b/B for rectangular HSS

γ = the chord slenderness ratio; the ratio of one-half the diameter to the wall thickness = $D/2t$ for round HSS; the ratio of one-half the width to wall thickness = $B/2t$ for rectangular HSS

η = the *load* length parameter, applicable only to rectangular HSS; the ratio of the length of contact of the branch with the chord in the plane of the connection to the chord width = N/B, where $N = H_b/\sin\theta$

θ = acute angle between the branch and chord (degrees)

2. Criteria for Round HSS

The interaction of *stress* due to *chord member forces* and local branch *connection forces* shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = 1.0 - 0.3U(1 + U) \quad \text{(K3-1)}$$

where U is the utilization ratio given by

$$U = |P_r/A_gF_c + M_r/SF_c| \quad \text{(K3-2)}$$

and

P_r = required axial strength in chord, kips (N).
M_r = required flexural strength in chord, kip-in. (N-mm)
A_g = chord gross area, in.^{2} (mm^2)
F_c = *available stress*, ksi (MPa)
S = chord elastic section modulus, in.^{3} (mm^3)

For design according to Section B3.3 (LRFD):

$P_r = P_u$ = required axial strength in chord, using *LRFD load combinations*, kips (N)
$M_r = M_u$ = required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)
$F_c = F_y$, ksi (MPa)

For design according to Section B3.4 (ASD):

$P_r = P_a$ = required axial strength in chord, using *ASD load combinations*, kips (N)
$M_r = M_a$ = required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)
$F_c = 0.6F_y$, ksi (MPa)

2a. Limits of Applicability

The criteria herein are applicable only when the connection configuration is within the following limits of applicability:

(1) Branch angle: $\theta \geq 30°$
(2) Chord wall slenderness: ratio of diameter to wall thickness less than or equal to 50 for *T*- and *Y-connections*; less than or equal to 40 for *cross-connections*
(3) Tension branch wall slenderness: ratio of diameter to wall thickness less than or equal to 50
(4) Compression branch wall slenderness: ratio of diameter to wall thickness less than or equal to $0.05E/F_y$
(5) Width ratio: $0.2 < D_b/D \leq 1.0$
(6) Strength: $F_y \leq 52$ ksi (360 MPa) for chord and branches
(7) Ductility: $F_y/F_u \leq 0.8$

2b. Branches with In-Plane Bending Moments in T-, Y- and Cross-Connections

The *design strength*, ϕM_n, and the *allowable strength*, M_n/Ω, shall be the lowest value obtained according to the *limit states* of *chord plastification* and *shear yielding (punching)*.

(a) For the limit state of chord plastification,

$$M_n \sin\theta = 5.39 F_y t^2 \gamma^{0.5} \beta\ D_b\ Q_f \tag{K3-3}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$M_n = 0.6F_y t D_b{}^2[(1 + 3\sin\theta)/4\sin^2\theta] \qquad \text{(K3-4)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

2c. Branches with Out-of-Plane Bending Moments in T-, Y- and Cross-Connections

The design strength, ϕM_n, and the allowable strength, M_n/Ω, shall be the lowest value obtained according to the limit states of chord plastification and shear yielding (punching).

(a) For the limit state of chord plastification,

$$M_n\sin\theta = F_y t^2 D_b[3.0/(1 - 0.81\beta)]Q_f \qquad \text{(K3-5)}$$

$$\phi = 0.90 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

(b) For the limit state of shear yielding (punching),

$$M_n = 0.6F_y t D_b{}^2[(3 + \sin\theta)/4\sin^2\theta]Q_f \qquad \text{(K3-6)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

This limit state need not be checked when $\beta > (1 - 1/\gamma)$.

2d. Branches with Combined Bending Moment and Axial Force in T-, Y- and Cross-Connections

Connections subject to branch axial load, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these *load effects*, should satisfy the following.

For design according to Section B3.3 (LRFD):

$$(P_r/\phi P_n) + (M_{r\text{-}ip}/\phi M_{n\text{-}ip})^2 + (M_{r\text{-}op}/\phi M_{n\text{-}op}) \leq 1.0 \qquad \text{(K3-7)}$$

where

P_r = P_u = required axial strength in branch, using LRFD load combinations, kips (N)

ϕP_n = design strength obtained from Section K2.2b

$M_{r\text{-}ip}$ = required in-plane flexural strength in branch, using LRFD load combinations, kip-in. (N-mm)

$\phi M_{n\text{-}ip}$ = design strength obtained from Section K3.2b

$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch, using LRFD load combinations, kip-in. (N-mm)

$\phi M_{n\text{-}op}$ = design strength obtained from Section K3.2c

For design according to Section B3.4 (ASD):

$$(P_r/(P_n/\Omega)) + (M_{r\text{-}ip}/(M_{n\text{-}ip}/\Omega))^2 + (M_{r\text{-}op}/(M_{n\text{-}op}/\Omega)) \leq 1.0 \qquad \text{(K3-8)}$$

where

P_r = P_a = required axial strength in branch, using ASD load combinations, kips (N)
P_n/Ω = *allowable strength* obtained from Section K2.2b
$M_{r\text{-}ip}$ = required in-plane flexural strength in branch, using ASD load combinations, kip-in. (N-mm)
$M_{n\text{-}ip}/\Omega$ = allowable strength obtained from Section K3.2b
$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch, using ASD load combinations, kip-in. (N-mm)
$M_{n\text{-}op}/\Omega$ = allowable strength obtained from Section K3.2c

3. Criteria for Rectangular HSS

The interaction of *stress* due to *chord member forces* and local branch *connection* forces shall be incorporated through the chord-stress interaction parameter Q_f.

When the chord is in tension,

$$Q_f = 1$$

When the chord is in compression,

$$Q_f = (1.3 - 0.4U/\beta) \leq 1 \tag{K3-9}$$

where U is the utilization ratio given by

$$U = |P_r/A_g F_c + M_r/SF_c| \tag{K3-10}$$

and

P_r = required axial strength in chord, kips (N)
M_r = required flexural strength in chord, kip-in. (N-mm)
A_g = chord gross area, in.^{2} (mm^2)
F_c = *available stress*, ksi, (MPa)
S = chord elastic section modulus, in.^{3} (mm^3)

For design according to Section B3.3 (LRFD):

P_r = P_u = required axial strength in chord, using *LRFD load combinations*, kips, (N)
M_r = M_u = required flexural strength in chord, using LRFD load combinations, kip-in. (N-mm)
F_c = F_y, ksi, (MPa)

For design according to Section B3.4 (ASD):

P_r = P_a = required axial strength in chord, using *ASD load combinations*, kips, (N)
M_r = M_a = required flexural strength in chord, using ASD load combinations, kip-in. (N-mm)
F_c = $0.6F_y$, ksi, (MPa)

3a. Limits of Applicability

The criteria herein are applicable only when the *connection* configuration is within the following limits:

(1) Branch angle is approximately 90°
(2) Chord wall slenderness: ratio of overall wall width to thickness less than or equal to 35
(3) Tension branch wall slenderness: ratio of overall wall width to thickness less than or equal to 35
(4) Compression branch wall slenderness: ratio of overall wall width to thickness less than or equal to $1.25(E/F_{yb})^{0.5}$ and also less than 35
(5) Width ratio: ratio of overall wall width of branch to overall wall width of chord greater than or equal to 0.25
(6) Aspect ratio: $0.5 \leq$ ratio of depth to width ≤ 2.0
(7) Strength: $F_y \leq 52$ ksi (360 MPa) for chord and branches
(8) Ductility: $F_y/F_u \leq 0.8$
(9) Other limits apply for specific criteria

3b. Branches with In-Plane Bending Moments in T- and Cross-Connections

The *design strength*, ϕM_n, and the *allowable strength*, M_n/Ω, shall be the lowest value obtained according to the *limit states* of *chord wall plastification*, sidewall *local yielding* and local yielding due to *uneven load distribution*.

(a) For the limit state of chord wall plastification,

$$M_n = F_y t^2 H_b[(1/2\eta) + 2/(1-\beta)^{0.5} + \eta/(1-\beta)]Q_f \qquad \text{(K3-11)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked when $\beta > 0.85$.

(b) For the limit state of sidewall local yielding,

$$M_n = 0.5F_y{}^*t(H_b + 5t)^2 \qquad \text{(K3-12)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

$F_y{}^* = F_y$ for *T-connections*
$F_y{}^* = 0.8F_y$ for *cross-connections*

This limit state need not be checked when $\beta < 0.85$.

(c) For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb}[Z_b - (1 - b_{eoi}/B_b)B_b H_b t_b] \qquad \text{(K3-13)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

b_{eoi} = $[10/(B/t)][F_y t/(F_{yb} t_b)]B_b \leq B_b$ (K3-14)
Z_b = branch plastic section modulus about the axis of bending, in.^{3}(mm^3)

This limit state need not be checked when $\beta < 0.85$.

3c. Branches with Out-of-Plane Bending Moments in T- and Cross-Connections

The design strength, ϕM_n, and the allowable strength, M_n/Ω, shall be the lowest value obtained according to the limit states of chord wall plastification, sidewall local yielding, local yielding due to uneven load distribution and chord *distortional failure.*

(a) For the limit state of chord wall plastification,

$$M_n = F_y t^2[0.5H_b(1+\beta)/(1-\beta) + [2BB_b(1+\beta)/(1-\beta)]^{0.5}]Q_f \quad \text{(K3-15)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked when $\beta > 0.85$.

(b) For the limit state of sidewall local yielding,

$$M_n = F_y{}^* t(B-t)(H_b+5t) \quad \text{(K3-16)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

where

$F_y{}^*$ = F_y for T-connections
$F_y{}^*$ = $0.8F_y$ for cross-connections

This limit state need not be checked when $\beta < 0.85$.

(c) For the limit state of local yielding due to uneven load distribution,

$$M_n = F_{yb}[Z_b - 0.5(1 - b_{eoi}/B_b)^2 B_b{}^2 t_b] \quad \text{(K3-17)}$$

$$\phi = 0.95 \text{ (LRFD)} \qquad \Omega = 1.58 \text{ (ASD)}$$

where

b_{eoi} = $[10/(B/t)][F_y t/(F_{yb} t_b)]B_b \leq B_b$ (K3-18)
Z_b = branch plastic section modulus about the axis of bending, in.^{3} (mm^3)

This limit state need not be checked when $\beta < 0.85$.

(d) For the limit state of chord distortional failure,

$$M_n = 2F_y t[H_b t + [BHt(B+H)]^{0.5}] \quad \text{(K3-19)}$$

$$\phi = 1.00 \text{ (LRFD)} \qquad \Omega = 1.50 \text{ (ASD)}$$

This limit state need not be checked for cross-connections or for T-connections if chord distortional failure is prevented by other means.

3d. Branches with Combined Bending Moment and Axial Force in T- and Cross-Connections

Connections subject to branch axial *load*, branch in-plane bending moment, and branch out-of-plane bending moment, or any combination of these *load effects*, should satisfy

For design according to Section B3.3 (LRFD)

$$(P_r/\phi P_n) + (M_{r\text{-}ip}/\phi M_{n\text{-}ip}) + (M_{r\text{-}op}/\phi M_{n\text{-}op}) \leq 1.0 \qquad \text{(K3-20)}$$

where

P_r = P_u = required axial strength in branch, using LRFD load combinations, kips (N)
ϕP_n = design strength obtained from Section K2.3b
$M_{r\text{-}ip}$ = required in-plane flexural strength in branch, using LRFD load combinations, kip-in. (N-mm)
$\phi M_{n\text{-}ip}$ = design strength obtained from Section K3.3b
$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch, using LRFD load combinations, kip-in. (N-mm)
$\phi M_{n\text{-}op}$ = design strength obtained from Section K3.3c

For design according to Section B3.4 (ASD)

$$(P_r/(P_n/\Omega)) + (M_{r\text{-}ip}/(M_{n\text{-}ip}/\Omega)) + (M_{r\text{-}op}/(M_{n\text{-}op}/\Omega)) \leq 1.0 \qquad \text{(K3-21)}$$

where

P_r = P_a = required axial strength in branch, using ASD load combinations, kips (N)
P_n/Ω = allowable strength obtained from Section K2.3b
$M_{r\text{-}ip}$ = required in-plane flexural strength in branch, using ASD load combinations, kip-in. (N-mm)
$M_{n\text{-}ip}/\Omega$ = allowable strength obtained from Section K3.3b
$M_{r\text{-}op}$ = required out-of-plane flexural strength in branch, using ASD load combinations, kip-in. (N-mm)
$M_{n\text{-}op}/\Omega$ = allowable strength obtained from Section K3.3c

CHAPTER L

DESIGN FOR SERVICEABILITY

This chapter addresses *serviceability* performance design requirements.

The chapter is organized as follows:

L1. General Provisions
L2. Camber
L3. Deflections
L4. Drift
L5. Vibration
L6. Wind-Induced Motion
L7. Expansion and Contraction
L8. Connection Slip

L1. GENERAL PROVISIONS

Serviceability is a state in which the function of a building, its appearance, maintainability, durability, and comfort of its occupants are preserved under normal usage. Limiting values of structural behavior for serviceability (for example, maximum deflections, accelerations) shall be chosen with due regard to the intended function of the structure. Serviceability shall be evaluated using appropriate *load combinations* for the serviceability *limit states* identified.

User Note: Additional information on serviceability limit states, service loads and appropriate load combinations for serviceability requirements can be found in ASCE 7, Appendix B and its Commentary. The performance requirements for serviceability in this chapter are consistent with those requirements. Service loads, as stipulated herein, are those that act on the structure at an arbitrary point in time. That is, the appropriate load combinations are often less severe than those in ASCE 7, Section 2.4, where the LRFD load combinations are given.

L2. CAMBER

Where *camber* is used to achieve proper position and location of the structure, the magnitude, direction and location of camber shall be specified in the structural drawings.

User Note: Camber recommendations are provided in the *Code of Standard Practice for Steel Buildings and Bridges*.

L3. DEFLECTIONS

Deflections in structural members and structural systems under appropriate *service load combinations* shall not impair the *serviceability* of the structure.

> **User Note:** Conditions to be considered include levelness of floors, alignment of structural members, integrity of building finishes, and other factors that affect the normal usage and function of the structure. For *composite* members, the additional deflections due to the shrinkage and creep of the concrete should be considered.

L4. DRIFT

Drift of a structure shall be evaluated under *service loads* to provide for *serviceability* of the structure, including the integrity of interior partitions and exterior *cladding*. *Drift* under strength *load combinations* shall not cause collision with adjacent structures or exceed the limiting values of such drifts that may be specified by the *applicable building code*.

L5. VIBRATION

The effect of vibration on the comfort of the occupants and the function of the structure shall be considered. The sources of vibration to be considered include pedestrian loading, vibrating machinery and others identified for the structure.

L6. WIND-INDUCED MOTION

The effect of wind-induced motion of buildings on the comfort of occupants shall be considered.

L7. EXPANSION AND CONTRACTION

The effects of thermal expansion and contraction of a building shall be considered. Damage to building *cladding* can cause water penetration and may lead to corrosion.

L8. CONNECTION SLIP

The effects of *connection* slip shall be included in the design where slip at bolted connections may cause deformations that impair the *serviceability* of the structure. Where appropriate, the connection shall be designed to preclude slip. For the design of slip-critical connections see Sections J3.8 and J3.9.

> **User Note:** For more information on connection slip, refer to the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

CHAPTER M

FABRICATION, ERECTION AND QUALITY CONTROL

This chapter addresses requirements for shop drawings, fabrication, shop painting, erection and *quality control.*

The chapter is organized as follows:

M1. Shop and Erection Drawings
M2. Fabrication
M3. Shop Painting
M4. Erection
M5. Quality Control

M1. SHOP AND ERECTION DRAWINGS

Shop drawings shall be prepared in advance of fabrication and give complete information necessary for the fabrication of the component parts of the structure, including the location, type and size of welds and bolts. Erection drawings shall be prepared in advance of erection and give information necessary for erection of the structure. Shop and erection drawings shall clearly distinguish between shop and field welds and bolts and shall clearly identify pretensioned and slip-critical high-strength bolted *connections*. Shop and erection drawings shall be made with due regard to speed and economy in fabrication and erection.

M2. FABRICATION

1. Cambering, Curving and Straightening

Local application of heat or mechanical means is permitted to be used to introduce or correct *camber*, curvature and straightness. The temperature of heated areas, as measured by approved methods, shall not exceed 1,100 °F (593 °C) for A514/A514M and A852/A852M steel nor 1,200 °F (649 °C) for other steels.

2. Thermal Cutting

Thermally cut edges shall meet the requirements of AWS D1.1, Sections 5.15.1.2, 5.15.4.3 and 5.15.4.4 with the exception that thermally cut free edges that will be subject to calculated static tensile *stress* shall be free of round-bottom *gouges* greater than $^3/_{16}$ in. (5 mm) deep and sharp V-shaped notches. *Gouges* deeper than $^3/_{16}$ in. (5 mm) and notches shall be removed by grinding or repaired by welding.

Reentrant corners, except reentrant corners of *beam copes* and weld access holes, shall meet the requirements of AWS D1.1, Section A5.16. If another specified contour is required it must be shown on the contract documents.

Beam copes and weld access holes shall meet the geometrical requirements of Section J1.6. Beam copes and weld access holes in shapes that are to be galvanized shall be ground. For shapes with a flange thickness not exceeding 2 in. (50 mm) the roughness of *thermally cut* surfaces of copes shall be no greater than a surface roughness value of 2,000 μin. (50 μm) as defined in ASME B46.1 Surface Texture (*Surface Roughness, Waviness, and Lay*). For beam copes and weld access holes in which the curved part of the access hole is thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and welded built-up shapes with material thickness greater than 2 in. (50 mm), a preheat temperature of not less than 150 °F (66 °C) shall be applied prior to thermal cutting. The thermally cut surface of access holes in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and built-up shapes with a material thickness greater than 2 in. (50 mm) shall be ground and inspected for cracks using magnetic particle inspection in accordance with ASTM E709. Any crack is unacceptable regardless of size or location.

User Note: The AWS Surface Roughness Guide for Oxygen Cutting (AWS C4.1-77) sample 3 may be used as a guide for evaluating the surface roughness of *copes* in shapes with flanges not exceeding 2 in. (50 mm) thick.

3. Planing of Edges

Planing or finishing of sheared or *thermally cut* edges of plates or shapes is not required unless specifically called for in the contract documents or included in a stipulated edge preparation for welding.

4. Welded Construction

The technique of welding, the workmanship, appearance and quality of welds, and the methods used in correcting nonconforming work shall be in accordance with AWS D1.1 except as modified in Section J2.

5. Bolted Construction

Parts of bolted members shall be pinned or bolted and rigidly held together during assembly. Use of a *drift* pin in bolt holes during assembly shall not distort the metal or enlarge the holes. Poor matching of holes shall be cause for rejection.

Bolt holes shall comply with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Section 3.3 except that *thermally cut* holes shall be permitted with a surface roughness profile not exceeding 1,000 μin. (25 μm) as defined in ASME B46.1. *Gouges* shall not exceed a depth of 1/16 in. (2 mm).

Fully inserted finger *shims*, with a total thickness of not more than 1/4 in. (6 mm) within a *joint*, are permitted in *joints* without changing the strength (based upon hole type) for the design of *connections*. The orientation of such *shims* is independent of the direction of application of the *load*.

The use of high-strength bolts shall conform to the requirements of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, except as modified in Section J3.

6. Compression Joints

Compression *joints* that depend on contact bearing as part of the *splice* strength shall have the bearing surfaces of individual fabricated pieces prepared by milling, sawing, or other suitable means.

7. Dimensional Tolerances

Dimensional tolerances shall be in accordance with the AISC *Code of Standard Practice for Steel Buildings and Bridges*.

8. Finish of Column Bases

Column bases and base plates shall be finished in accordance with the following requirements:

(1) Steel bearing plates 2 in. (50 mm) or less in thickness are permitted without milling, provided a satisfactory contact bearing is obtained. Steel bearing plates over 2 in. (50 mm) but not over 4 in. (100 mm) in thickness are permitted to be straightened by pressing or, if presses are not available, by milling for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section), to obtain a satisfactory contact bearing. Steel bearing plates over 4 in. (100 mm) in thickness shall be milled for bearing surfaces (except as noted in subparagraphs 2 and 3 of this section).

(2) Bottom surfaces of bearing plates and *column* bases that are grouted to ensure full bearing contact on foundations need not be milled.

(3) Top surfaces of bearing plates need not be milled when complete-*joint*-penetration *groove welds* are provided between the *column* and the bearing plate.

9. Holes for Anchor Rods

Holes for anchor rods shall be permitted to be *thermally cut* in accordance with the provisions of Section M2.2.

10. Drain Holes

When water can collect inside *HSS* or box members, either during construction or during service, the member shall be sealed, provided with a drain hole at the base, or protected by other suitable means.

11. Requirements for Galvanized Members

Members and parts to be galvanized shall be designed, detailed and fabricated to provide for flow and drainage of pickling fluids and zinc and to prevent pressure build-up in enclosed parts.

> **User Note:** See *The Design of Products to be Hot-Dip Galvanized After Fabrication*, American Galvanizer's Association, and ASTM A123, A153, A384 and A780 for useful information on design and detailing of galvanized members.

M3. SHOP PAINTING

1. General Requirements

Shop painting and surface preparation shall be in accordance with the provisions of the AISC *Code of Standard Practice for Steel Buildings and Bridges*.

Shop paint is not required unless specified by the contract documents.

2. Inaccessible Surfaces

Except for contact surfaces, surfaces inaccessible after shop assembly shall be cleaned and painted prior to assembly, if required by the design documents.

3. Contact Surfaces

Paint is permitted in *bearing-type connection*s. For *slip-critical connection*s, the *faying surface* requirements shall be in accordance with the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, Section 3.2.2(b).

4. Finished Surfaces

Machine-finished surfaces shall be protected against corrosion by a rust inhibitive coating that can be removed prior to erection, or which has characteristics that make removal prior to erection unnecessary.

5. Surfaces Adjacent to Field Welds

Unless otherwise specified in the design documents, surfaces within 2 in. (50 mm) of any field weld location shall be free of materials that would prevent proper welding or produce objectionable fumes during welding.

M4. ERECTION

1. Alignment of Column Bases

Column bases shall be set level and to correct elevation with full bearing on concrete or masonry.

2. Bracing

The frame of steel skeleton buildings shall be carried up true and plumb within the limits defined in the AISC *Code of Standard Practice for Steel Buildings and*

Bridges. Temporary bracing shall be provided, in accordance with the requirements of the *Code of Standard Practice for Steel Buildings and Bridges*, wherever necessary to support the loads to which the structure may be subjected, including equipment and the operation of same. Such bracing shall be left in place as long as required for safety.

3. Alignment

No permanent bolting or welding shall be performed until the adjacent affected portions of the structure have been properly aligned.

4. Fit of Column Compression Joints and Base Plates

Lack of contact bearing not exceeding a gap of 1/16 in. (2 mm), regardless of the type of *splice* used (*partial-joint-penetration groove welded* or bolted), is permitted. If the gap exceeds 1/16 in. (2 mm), but is less than 1/4 in. (6 mm), and if an engineering investigation shows that sufficient contact area does not exist, the gap shall be packed out with nontapered steel *shim*s. Shims need not be other than mild steel, regardless of the grade of the main material.

5. Field Welding

Shop paint on surfaces adjacent to *joint*s to be field welded shall be wire brushed if necessary to assure weld quality.

Field welding of attachments to installed embedments in contact with concrete shall be done in such a manner as to avoid excessive thermal expansion of the embedment which could result in spalling or cracking of the concrete or excessive *stress* in the embedment anchors.

6. Field Painting

Responsibility for touch-up painting, cleaning and field painting shall be allocated in accordance with accepted local practices, and this allocation shall be set forth explicitly in the design documents.

7. Field Connections

As erection progresses, the structure shall be securely bolted or welded to support the dead, wind and erection *load*s.

M5. QUALITY CONTROL

The fabricator shall provide *quality control* procedures to the extent that the fabricator deems necessary to assure that the work is performed in accordance with this Specification. In addition to the fabricator's quality control procedures, material and workmanship at all times may be subject to inspection by qualified inspectors representing the purchaser. If such inspection by representatives of the purchaser will be required, it shall be so stated in the design documents.

1. Cooperation

As far as possible, the inspection by representatives of the purchaser shall be made at the fabricator's plant. The fabricator shall cooperate with the inspector, permitting access for inspection to all places where work is being done. The purchaser's inspector shall schedule this work for minimum interruption to the work of the fabricator.

2. Rejections

Material or workmanship not in conformance with the provisions of this Specification may be rejected at any time during the progress of the work.

The fabricator shall receive copies of all reports furnished to the purchaser by the inspection agency.

3. Inspection of Welding

The inspection of welding shall be performed in accordance with the provisions of AWS D1.1 except as modified in Section J2.

When visual inspection is required to be performed by AWS certified welding inspectors, it shall be so specified in the design documents.

When nondestructive testing is required, the process, extent and standards of acceptance shall be clearly defined in the design documents.

4. Inspection of Slip-Critical High-Strength Bolted Connections

The inspection of slip-critical high-strength bolted *connection*s shall be in accordance with the provisions of the RCSC *Specification for Structural Joints Using ASTM A325 or A490 Bolts*.

5. Identification of Steel

The fabricator shall be able to demonstrate by a written procedure and by actual practice a method of material identification, visible at least through the "fit-up" operation, for the main structural elements of each shipping piece.

APPENDIX 1

INELASTIC ANALYSIS AND DESIGN

Design by *inelastic analysis* is subject to the supplementary provisions of this appendix.

The appendix is organized as follows:

1.1. General Provisions
1.2. Materials
1.3. Moment Redistribution
1.4. Local Buckling
1.5. Stability and Second-Order Effects
1.6. Columns and Other Compression Members
1.7. Beams and Other Flexural Members
1.8. Members under Combined Forces
1.9. Connections

1.1. GENERAL PROVISIONS

Inelastic analysis is permitted for design according to the provisions of Section B3.3 (LRFD). Inelastic analysis is not permitted for design according to the provisions of Section B3.4 (ASD) except as provided in Section 1.3.

1.2. MATERIALS

Members undergoing plastic hinging shall have a *specified minimum yield stress* not exceeding 65 ksi (450 MPa).

1.3. MOMENT REDISTRIBUTION

*Beam*s and girders composed of *compact sections* as defined in Section B4 and satisfying the *unbraced length* requirements of Section 1.7, including *composite* members, may be proportioned for nine-tenths of the negative moments at points of support, produced by the *gravity loading* computed by an *elastic analysis*, provided that the maximum positive moment is increased by one-tenth of the average negative moments. This reduction is not permitted for moments produced by loading on cantilevers and for design according to Sections 1.4 through 1.8 of this appendix.

If the negative moment is resisted by a *column* rigidly framed to the beam or girder, the one-tenth reduction may be used in proportioning the column for combined axial *force* and flexure, provided that the axial force does not exceed $0.15\phi_c F_y A_g$ for LRFD or $0.15F_y A_g/\Omega_c$ for ASD,

where

A_g = gross area of member, in.^{2} (mm^2)
F_y = *specified minimum yield stress* of the compression flange, ksi (MPa)

ϕ_c = *resistance factor* for compression = 0.90
Ω_c = *safety factor* for compression = 1.67

1.4. LOCAL BUCKLING

Flanges and webs of members subject to plastic hinging in combined flexure and axial compression shall be compact with width-thickness ratios less than or equal to the limiting λ_p defined in Table B4.1 or as modified as follows:

(a) For webs of doubly symmetric wide flange members and rectangular *HSS* in combined flexure and compression

(i) For $P_u/\phi_b P_y \le 0.125$

$$h/t_w \le 3.76\sqrt{\frac{E}{F_y}}\left(1 - \frac{2.75P_u}{\phi_b P_y}\right) \quad \text{(A-1-1)}$$

(ii) For $P_u/\phi_b P_y > 0.125$

$$h/t_w \le 1.12\sqrt{\frac{E}{F_y}}\left(2.33 - \frac{P_u}{\phi_b P_y}\right) \ge 1.49\sqrt{\frac{E}{F_y}} \quad \text{(A-1-2)}$$

where

E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
F_y = specified minimum yield stress of the type of steel being used, ksi (MPa)
h = as defined in Section B4.2, in. (mm)
P_u = required axial strength in compression, kips (N)
P_y = member yield strength, kips (N)
t_w = web thickness, in. (mm)
ϕ_b = resistance factor for flexure = 0.90

(b) For flanges of rectangular box and hollow structural sections of uniform thickness subject to bending or compression, flange *cover plates*, and *diaphragm plates* between lines of *fasteners* or welds

$$b/t \le 0.94\sqrt{E/F_y} \quad \text{(A-1-3)}$$

where

b = as defined in Section B4.2, in. (mm)
t = as defined in Section B4.2, in. (mm)

(c) For circular hollow sections in flexure

$$D/t \le 0.045E/F_y \quad \text{(A-1-4)}$$

where

D = outside diameter of round HSS member, in. (mm)

1.5. STABILITY AND SECOND-ORDER EFFECTS

Continuous *beams* not subjected to axial *loads* and that do not contribute to lateral *stability* of framed structures may be designed based on a *first-order inelastic analysis* or a plastic *mechanism* analysis.

Braced frames and *moment frames* may be designed based on a *first-order inelastic analysis* or a plastic *mechanism* analysis provided that *stability* and *second-order effects* are taken into account.

Structures may be designed on the basis of a second-order *inelastic analysis*. For *beam-columns*, *connections* and connected members, the *required strengths* shall be determined from a second-order inelastic analysis, where equilibrium is satisfied on the deformed geometry, taking into account the change in *stiffness* due to yielding.

1. Braced Frames

In *braced frames* designed on the basis of *inelastic analysis*, braces shall be designed to remain elastic under the *design loads*. The required axial strength for *columns* and compression braces shall not exceed $\phi_c\ (0.85 F_y A_g)$,

where

$$\phi_c = 0.90\,(\mathrm{LRFD})$$

2. Moment Frames

In *moment frames* designed on the basis of *inelastic analysis*, the required axial strength of columns shall not exceed $\phi_c(0.75 F_y A_g)$,

where

$$\phi_c = 0.90\,(\mathrm{LRFD})$$

1.6. COLUMNS AND OTHER COMPRESSION MEMBERS

In addition to the limits set in Sections 1.5.1 and 1.5.2, the required axial strength of *columns* designed on the basis of *inelastic analysis* shall not exceed the design strength, $\phi_c P_n$, determined according to the provisions of Section E3.

Design by inelastic analysis is permitted if the column slenderness ratio, L/r, does not exceed $4.71\sqrt{E/F_y}$,

where

L = laterally unbraced length of a member, in. (mm)
r = governing radius of gyration, in. (mm)

User Note: A well-proportioned member will not be expected to reach this limit.

1.7. BEAMS AND OTHER FLEXURAL MEMBERS

The required moment strength, M_u, of *beams* designed on the basis of *inelastic analysis* shall not exceed the *design strength*, ϕM_n, where

$$M_n = M_p = F_y Z < 1.6 F_y S \qquad \text{(A-1-6)}$$

$$\phi = 0.90\,(\text{LRFD})$$

Design by inelastic analysis is permitted for members that are compact as defined in Section B4 and as modified in Section 1.4.

The laterally *unbraced length*, L_b,of the compression flange adjacent to *plastic hinge* locations shall not exceed L_{pd}, determined as follows.

(a) For doubly symmetric and singly symmetric I-shaped members with the compression flange equal to or larger than the tension flange loaded in the plane of the web:

$$L_{pd} = \left[0.12 + 0.076\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \qquad \text{(A-1-7)}$$

where

M_1 = smaller moment at end of unbraced length of beam, kip-in. (N-mm)
M_2 = larger moment at end of unbraced length of beam, kip-in. (N-mm)
r_y = radius of gyration about minor axis, in. (mm)

(M_1/M_2) is positive when moments cause *reverse curvature* and negative for *single curvature*.

(b) For solid rectangular bars and symmetric box beams:

$$L_{pd} = \left[0.17 + 0.10\left(\frac{M_1}{M_2}\right)\right]\left(\frac{E}{F_y}\right) r_y \geq 0.10\left(\frac{E}{F_y}\right) r_y \qquad \text{(A-1-8)}$$

There is no limit on L_b for members with circular or square cross sections or for any beam bent about its minor axis.

1.8. MEMBERS UNDER COMBINED FORCES

When inelastic analysis is used for symmetric members subject to bending and axial force, the provisions in Section H1 apply.

Inelastic analysis is not permitted for members subject to torsion and combined torsion, flexure, shear and/or axial force.

1.9. CONNECTIONS

Connections adjacent to plastic hinging regions of connected members shall be designed with sufficient strength and ductility to sustain the *forces* and deformations imposed under the required *loads*.

APPENDIX 2

DESIGN FOR PONDING

This appendix provides methods for determining whether a roof system has adequate strength and stiffness to resist ponding.

The appendix is organized as follows:

2.1. Simplified Design for Ponding
2.2. Improved Design for Ponding

2.1. SIMPLIFIED DESIGN FOR PONDING

The roof system shall be considered stable for *ponding* and no further investigation is needed if both of the following two conditions are met:

$$C_p + 0.9C_s \leq 0.25 \tag{A-2-1}$$

$$I_d \geq 25(S^4)10^{-6} \tag{A-2-2}$$

$$I_d \geq 3940\, S^4 \text{ (S.I.)} \tag{A-2-2M}$$

where

$$C_p = \frac{32L_sL_p^4}{10^7 I_p}$$

$$C_p = \frac{504L_sL_p^4}{I_p} \text{ (S.I.)}$$

$$C_s = \frac{32SL_s^4}{10^7 I_s}$$

$$C_s = \frac{504SL_s^4}{I_s} \text{ (S.I.)}$$

L_p = *column* spacing in direction of girder (length of primary members), ft (m)
L_s = column spacing perpendicular to direction of girder (length of secondary members), ft (m)
S = spacing of secondary members, ft (m)
I_p = moment of inertia of primary members, in.^{4} (mm^4)
I_s = moment of inertia of secondary members, in.^{4} (mm^4)
I_d = moment of inertia of the steel deck supported on secondary members, in.^{4}per ft (mm^4 per m)

For trusses and steel joists, the moment of inertia I_s shall be decreased 15 percent when used in the above equation. A steel deck shall be considered a secondary member when it is directly supported by the primary members.

2.2. IMPROVED DESIGN FOR PONDING

The provisions given below are permitted to be used when a more exact determination of framing *stiffness* is needed than that given in Section 2.1.

For primary members, the stress index shall be

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \tag{A-2-3}$$

For secondary members, the stress index shall be

$$U_s = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \tag{A-2-4}$$

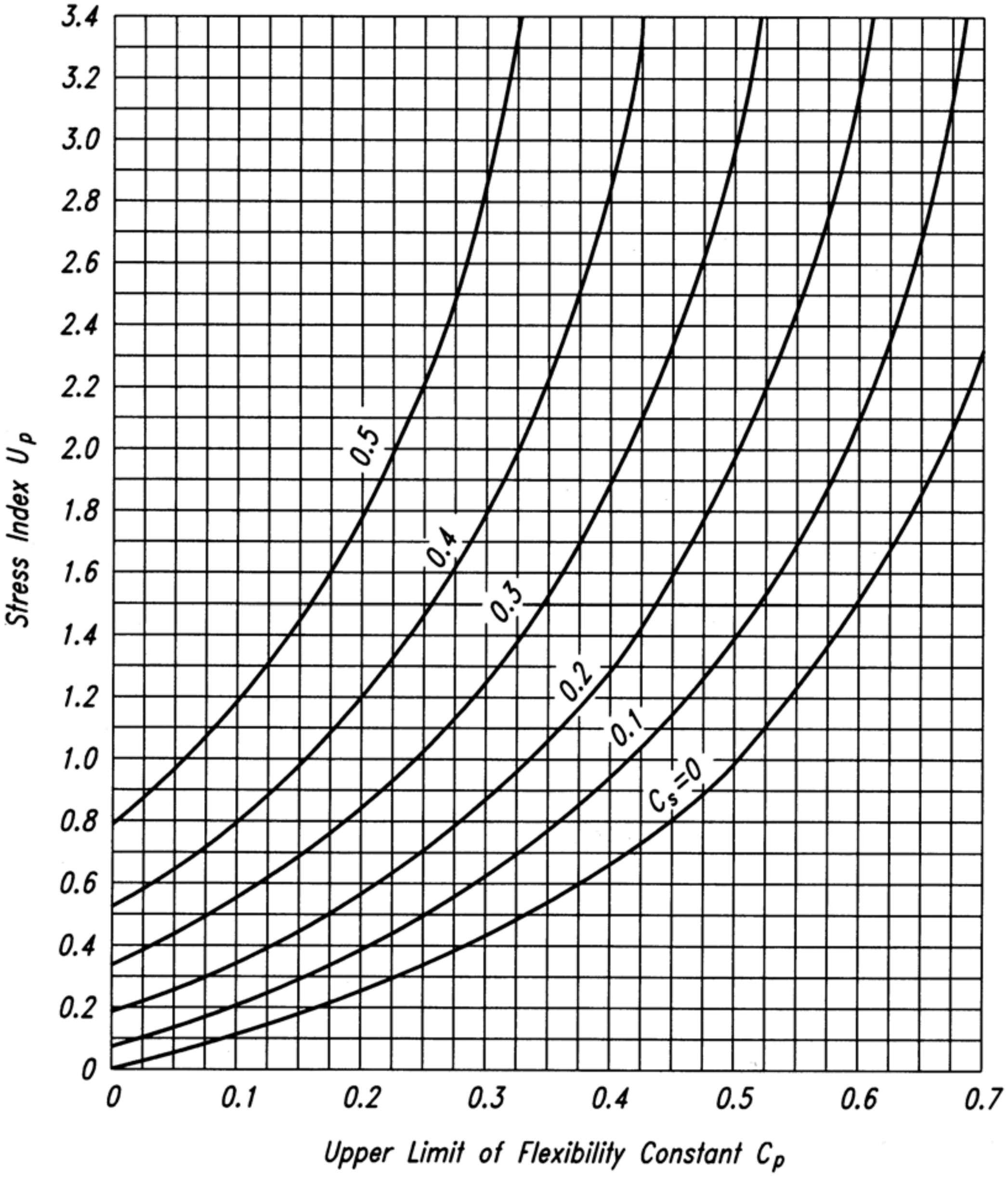


Fig. A-2-1. Limiting flexibility coefficient for the primary systems.

where

f_o = *stress* due to the load combination $(D + R)$
D = nominal dead *load*
R = *nominal load* due to rainwater or snow, exclusive of the *ponding* contribution, ksi (MPa)

For roof framing consisting of primary and secondary members, the combined stiffness shall be evaluated as follows: enter Figure A-2-1 at the level of the computed stress index U_p determined for the primary beam; move horizontally to the computed $C_s$value of the secondary beams and then downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is more than the value of C_p computed for the given primary member; if not, a stiffer primary or secondary beam, or combination of both, is required.

A similar procedure must be followed using Figure A-2-2.

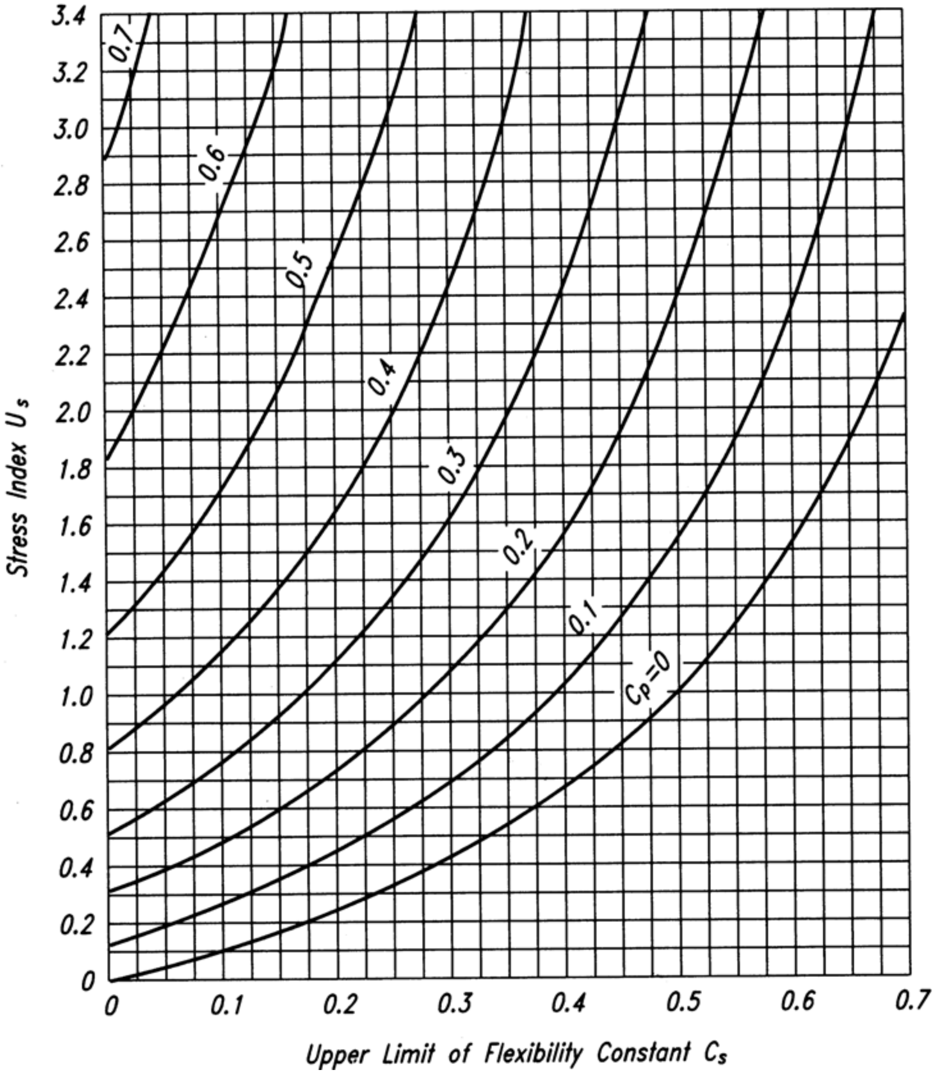


Fig. A-2-2. Limiting flexibility coefficient for the secondary systems.

For roof framing consisting of a series of equally spaced wall-bearing beams, the stiffness shall be evaluated as follows. The beams are considered as secondary members supported on an infinitely stiff primary member. For this case, enter Figure A-2-2 with the computed *stress* index U_s. The limiting value of C_s is determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

User Note: The *ponding* deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia (per foot (meter) of width normal to its span) to $0.000025l^4$ in.^{4}/ft ($3940l^4$ mm^4/m).

For roof framing consisting of metal deck spanning between beams supported on columns, the stiffness shall be evaluated as follows. Employ Figure A-2-1 or A-2-2 using as C_s the flexibility constant for a 1 ft (1 m) width of the roof deck ($S = 1.0$).

APPENDIX 3

DESIGN FOR FATIGUE

This appendix applies to members and *connections* subject to high cycle loading within the elastic range of stresses of frequency and magnitude sufficient to initiate cracking and progressive failure, which defines the *limit state* of *fatigue*.

The appendix is organized as follows:

3.1. General
3.2. Calculation of Maximum Stresses and Stress Ranges
3.3. Design Stress Range
3.4. Bolts and Threaded Parts
3.5. Special Fabrication and Erection Requirements

3.1. GENERAL

The provisions of this Appendix apply to stresses calculated on the basis of *service loads*. The maximum permitted *stress* due to unfactored *loads* is $0.66F_y$.

Stress range is defined as the magnitude of the change in stress due to the application or removal of the service live load. In the case of a stress reversal, the stress range shall be computed as the numerical sum of maximum repeated tensile and compressive stresses or the numerical sum of maximum shearing stresses of opposite direction at the point of probable crack initiation.

In the case of complete-joint-penetration butt welds, the maximum *design stress range* calculated by Equation A-3-1 applies only to welds with internal soundness meeting the acceptance requirements of Section 6.12.2 or 6.13.2 of AWS D1.1.

No evaluation of *fatigue* resistance is required if the live load stress range is less than the threshold stress range, F_{TH}. See Table A-3.1.

No evaluation of fatigue resistance is required if the number of cycles of application of live load is less than 20,000.

The cyclic load resistance determined by the provisions of this Appendix is applicable to structures with suitable corrosion protection or subject only to mildly corrosive atmospheres, such as normal atmospheric conditions.

The cyclic load resistance determined by the provisions of this Appendix is applicable only to structures subject to temperatures not exceeding 300 °F (150 °C).

The *engineer of record* shall provide either complete details including weld sizes or shall specify the planned cycle life and the maximum range of moments, shears and reactions for the *connections*.

3.2. CALCULATION OF MAXIMUM STRESSES AND STRESS RANGES

Calculated stresses shall be based upon *elastic analysis*. Stresses shall not be amplified by *stress concentration* factors for geometrical discontinuities.

For bolts and threaded rods subject to axial tension, the calculated stresses shall include the effects of *prying action*, if any. In the case of axial stress combined with bending, the maximum stresses, of each kind, shall be those determined for concurrent arrangements of the applied *load*.

For members having symmetric cross sections, the *fasteners* and welds shall be arranged symmetrically about the axis of the member, or the total stresses including those due to eccentricity shall be included in the calculation of the *stress* range.

For axially loaded angle members where the center of gravity of the connecting welds lies between the line of the center of gravity of the angle cross section and the center of the connected leg, the effects of eccentricity shall be ignored. If the center of gravity of the connecting welds lies outside this zone, the total stresses, including those due to *joint eccentricity*, shall be included in the calculation of *stress* range.

3.3. DESIGN STRESS RANGE

The range of *stress* at *service loads* shall not exceed the *design stress range* computed as follows.

(a) For stress categories A, B, B′, C, D, E and E′ the design stress range, F_{SR}, shall be determined by Equation A-3-1 or A-3-1M.

$$F_{SR} = \left(\frac{C_f}{N}\right)^{0.333} \geq F_{TH} \tag{A-3-1}$$

$$F_{SR} = \left(\frac{C_f \times 329}{N}\right)^{0.333} \geq F_{TH} \quad \text{(S.I.)} \tag{A-3-1M}$$

where

F_{SR} = design stress range, ksi (MPa)

C_f = constant from Table A-3.1 for the category

N = number of stress range fluctuations in design life

= number of stress range fluctuations per day × 365 × years of design life

F_{TH} = threshold *fatigue stress* range, maximum *stress* range for indefinite design life from Table A-3.1, ksi (MPa)

(b) For stress category F, the design stress range, F_{SR}, shall be determined by Equation A-3-2 or A-3-2M.

$$F_{SR} = \left(\frac{C_f}{N}\right)^{0.167} \geq F_{TH} \qquad \text{(A-3-2)}$$

$$F_{SR} = \left(\frac{C_f \times 11 \times 10^4}{N}\right)^{0.167} \geq F_{TH} \quad \text{(S.I.)} \qquad \text{(A-3-2M)}$$

(c) For tension-loaded plate elements connected at their end by cruciform, T, or corner details with *complete-joint-penetration* (CJP) *groove welds* or *partial-joint-penetration* (PJP) *groove welds*, *fillet welds*, or combinations of the preceding, transverse to the direction of stress, the design stress range on the cross section of the tension-loaded plate element at the toe of the weld shall be determined as follows:

(i) Based upon crack initiation from the toe of the weld on the tension loaded plate element the design stress range, F_{SR}, shall be determined by Equation A-3-3 or A-3-3M, for stress category C which is equal to

$$F_{SR} = \left(\frac{44 \times 10^8}{N}\right)^{0.333} \geq 10 \qquad \text{(A-3-3)}$$

$$F_{SR} = \left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \geq 68.9 \quad \text{(S.I.)} \qquad \text{(A-3-3M)}$$

(ii) Based upon crack initiation from the root of the weld the design stress range, F_{SR}, on the tension loaded plate element using transverse PJP groove welds, with or without reinforcing or contouring fillet welds, the design stress range on the cross section at the toe of the weld shall be determined by Equation A-3-4 or A-3-4M, stress category C′ as follows:

$$F_{SR} = R_{PJP}\left(\frac{44 \times 10^8}{N}\right)^{0.333} \qquad \text{(A-3-4)}$$

$$F_{SR} = R_{PJP}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \quad \text{(S.I.)} \qquad \text{(A-3-4M)}$$

where

R_{PJP} is the reduction factor for reinforced or nonreinforced transverse PJP groove welds determined as follows:

$$R_{PJP} = \left(\frac{0.65 - 0.59\left(\frac{2a}{t_p}\right) + 0.72\left(\frac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0$$

$$R_{PJP} = \left(\frac{1.12 - 1.01\left(\frac{2a}{t_p}\right) + 1.24\left(\frac{w}{t_p}\right)}{t_p^{0.167}}\right) \leq 1.0 \qquad \text{(S.I.)}$$

If $R_{PJP} = 1.0$, use stress category C.

$2a$ = the length of the nonwelded root face in the direction of the thickness of the tension-loaded plate, in. (mm)

w = the leg size of the reinforcing or contouring fillet, if any, in the direction of the thickness of the tension-loaded plate, in. (mm)

t_p = thickness of tension loaded plate, in. (mm)

(iii) Based upon crack initiation from the roots of a pair of transverse fillet welds on opposite sides of the tension loaded plate element the design stress range, F_{SR}, on the cross section at the toe of the welds shall be determined by Equation A-3-5 or A-3-5M, stress category C″ as follows:

$$F_{SR} = R_{FIL}\left(\frac{44 \times 10^8}{N}\right)^{0.333} \qquad \text{(A-3-5)}$$

$$F_{SR} = R_{FIL}\left(\frac{14.4 \times 10^{11}}{N}\right)^{0.333} \quad \text{(S.I.)} \qquad \text{(A-3-5M)}$$

where

R_{FIL} is the reduction factor for joints using a pair of transverse fillet welds only.

$$R_{FIL} = \left(\frac{0.06 + 0.72\left(w/t_p\right)}{t_p^{0.167}}\right) \leq 1.0$$

$$R_{FIL} = \left(\frac{0.10 + 1.24\left(w/t_p\right)}{t_p^{0.167}}\right) \leq 1.0 \quad \text{(S.I.)}$$

If $R_{FIL} = 1.0$, use stress category C.

3.4. BOLTS AND THREADED PARTS

The range of *stress* at *service loads* shall not exceed the stress range computed as follows.

(a) For mechanically fastened *connections* loaded in shear, the maximum range of stress in the connected material at service loads shall not exceed the *design stress range* computed using Equation A-3-1 where C_f and F_{TH} are taken from Section 2 of Table A-3.1.

(b) For high-strength bolts, common bolts, and threaded anchor rods with cut, ground or rolled threads, the maximum range of tensile stress on the net tensile area from applied axial *load* and moment plus load due to *prying action* shall not exceed the design stress range computed using Equation A-3-1 or A-3-1M. The factor C_f shall be taken as 3.9×10^8 (as for stress category E′). The threshold stress, F_{TH} shall be taken as 7 ksi (48 MPa) (as for stress category D). The net tensile area is given by Equation A-3-6 and A-3-6M.

$$A_t = \frac{\pi}{4}\left(d_b - \frac{0.9743}{n}\right)^2 \qquad \text{(A-3-6)}$$

$$A_t = \frac{\pi}{4}\left(d_b - 0.9382P\right)^2 \quad \text{(S.I.)} \qquad \text{(A-3-6M)}$$

where

P = *pitch*, in. per thread (mm per thread)
d_b = the nominal diameter (body or shank diameter), in. (mm)
n = threads per in. (threads per mm)

For *joints* in which the material within the *grip* is not limited to steel or joints which are not tensioned to the requirements of Table J3.1 or J3.1M, all axial load and moment applied to the joint plus effects of any *prying action* shall be assumed to be carried exclusively by the bolts or rods.

For joints in which the material within the grip is limited to steel and which are tensioned to the requirements of Table J3.1 or J3.1M, an analysis of the relative *stiffness* of the connected parts and bolts shall be permitted to be used to determine the tensile *stress* range in the pretensioned bolts due to the total service live load and moment plus effects of any prying action. Alternatively, the stress range in the bolts shall be assumed to be equal to the stress on the net tensile area due to 20 percent of the absolute value of the service load axial load and moment from dead, live and other loads.

3.5. SPECIAL FABRICATION AND ERECTION REQUIREMENTS

Longitudinal backing bars are permitted to remain in place, and if used, shall be continuous. If splicing is necessary for long *joints*, the bar shall be joined with complete penetration butt joints and the reinforcement ground prior to assembly in the joint.

In transverse joints subject to tension, backing bars, if used, shall be removed and the joint back gouged and welded.

In transverse complete-joint-penetration T and corner joints, a reinforcing *fillet weld*, not less than $^1/_4$ in. (6 mm) in size shall be added at re-entrant corners.

The surface roughness of flame cut edges subject to significant cyclic tensile *stress* ranges shall not exceed 1,000 μin. (25 μm), where ASME B46.1 is the reference standard.

Reentrant corners at cuts, *copes* and weld access holes shall form a radius of not less than $^3/_8$ in. (10 mm) by predrilling or subpunching and reaming a hole, or by thermal cutting to form the radius of the cut. If the radius portion is formed by thermal cutting, the cut surface shall be ground to a bright metal surface.

For transverse butt joints in regions of high tensile stress, run-off tabs shall be used to provide for cascading the weld termination outside the finished joint. End dams shall not be used. Run-off tabs shall be removed and the end of the weld finished flush with the edge of the member.

See Section J2.2b for requirements for *end returns* on certain fillet welds subject to cyclic *service loading*.

TABLE A-3.1
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING				
1.1 Base metal, except non-coated weathering steel, with rolled or cleaned surface. Flame-cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without reentrant corners.	A	250×10^8	24 (165)	Away from all welds or structural *connections*
1.2 Non-coated weathering steel base metal with rolled or cleaned surface. Flame-cut edges with surface roughness value of 1,000 μin. (25 μm) or less, but without reentrant corners.	B	120×10^8	16 (110)	Away from all welds or structural *connections*
1.3 Member with drilled or reamed holes. Member with re-entrant corners at *copes*, cuts, block-outs or other geometrical discontinuities made to requirements of Appendix 3.5, except weld access holes.	B	120×10^8	16 (110)	At any external edge or at hole perimeter
1.4 Rolled cross sections with weld access holes made to requirements of Section J1.6 and Appendix 3.5. Members with drilled or reamed holes containing bolts for attachment of light bracing where there is a small longitudinal component of brace *force*.	C	44×10^8	10 (69)	At *reentrant* corner of weld access hole or at any small hole (may contain bolt for minor *connections*)
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS				
2.1 Gross area of base metal in lap joints connected by high-strength bolts in joints satisfying all requirements for slip-critical connections.	B	120×10^8	16 (110)	Through gross section near hole
2.2 Base metal at net section of high-strength bolted joints, designed on the basis of bearing resistance, but fabricated and installed to all requirements for slip-critical connections.	B	120×10^8	16 (110)	In net section originating at side of hole
2.3 Base metal at the net section of other mechanically fastened joints except eye bars and pin plates.	D	22×10^8	7 (48)	In net section originating at side of hole
2.4 Base metal at net section of *eyebar* head or pin plate.	E	11×10^8	4.5 (31)	In net section originating at side of hole

TABLE A-3.1 (cont.)
Fatigue Design Parameters

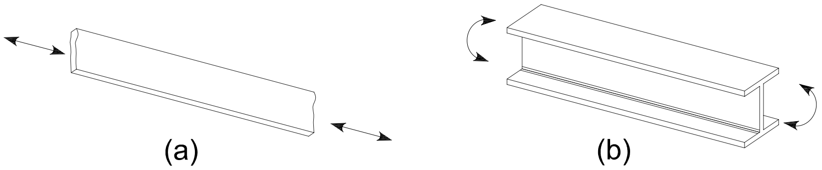
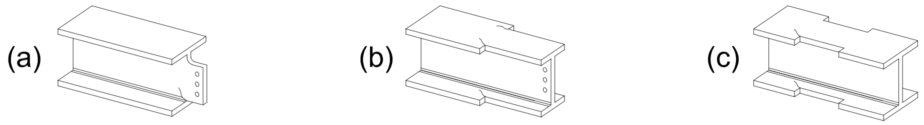
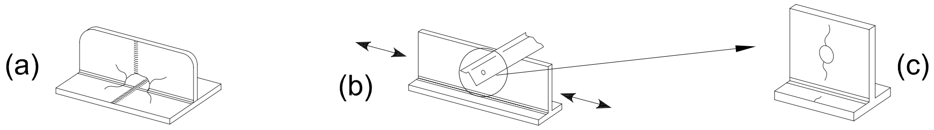
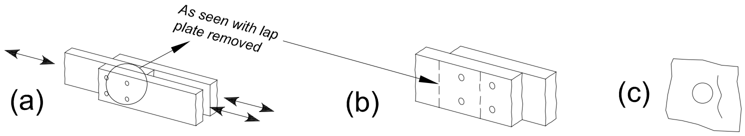
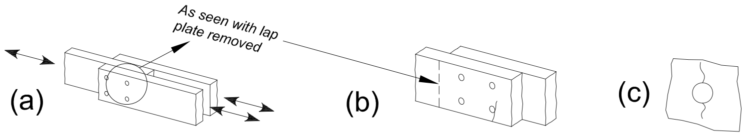
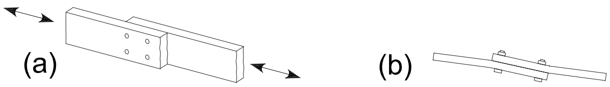
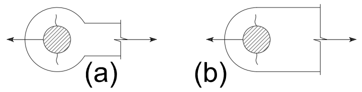
	Illustrative Typical Examples
SECTION 1 – PLAIN MATERIAL AWAY FROM ANY WELDING	
1.1 and 1.2	
1.3	
1.4	
SECTION 2 – CONNECTED MATERIAL IN MECHANICALLY FASTENED JOINTS	
2.1	
2.2	
2.3	
2.4	

TABLE A-3.1 (cont.)
Fatigue Design Parameters

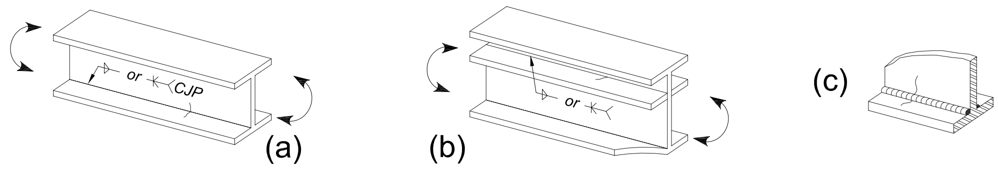
Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS				
3.1 Base metal and weld metal in members without attachments built-up of plates or shapes connected by continuous longitudinal complete-joint-penetration groove welds, back gouged and welded from second side, or by continuous fillet welds.	B	120×10^8	16 (110)	From surface or internal discontinuities in weld away from end of weld
3.2 Base metal and weld metal in members without attachments built-up of plates or shapes, connected by continuous longitudinal complete-joint-penetration groove welds with backing bars not removed, or by continuous partial-joint-penetration groove welds.	B′	61×10^8	12 (83)	From surface or internal discontinuities in weld, including weld attaching backing bars
3.3 Base metal and weld metal termination of longitudinal welds at weld access holes in connected built-up members.	D	22×10^8	7 (48)	From the weld termination into the web or flange
3.4 Base metal at ends of longitudinal intermittent fillet weld segments.	E	11×10^8	4.5 (31)	In connected material at start and stop locations of any weld deposit
3.5 Base metal at ends of partial length welded coverplates narrower than the flange having square or tapered ends, with or without welds across the ends of coverplates wider than the flange with welds across the ends.				In flange at toe of end weld or in flange at termination of longitudinal weld or in edge of flange with wide coverplates
Flange thickness $\leq$ 0.8 in. (20 mm)	E	11×10^8	4.5 (31)	
Flange thickness $>$ 0.8 in. (20 mm)	E′	3.9×10^8	2.6 (18)	
3.6 Base metal at ends of partial length welded coverplates wider than the flange without welds across the ends.	E′	3.9×10^8	2.6 (18)	In edge of flange at end of coverplate weld
SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS				
4.1 Base metal at junction of axially loaded members with longitudinally welded end connections. Welds shall be on each side of the axis of the member to balance weld stresses.				Initiating from end of any weld termination extending into the base metal
$t \leq$ 0.8 in. (20 mm)	E	11×10^8	4.5 (31)	
$t >$ 0.8 in. (20 mm)	E′	3.9×10^8	2.6 (18)	

TABLE A-3.1 (cont.)
Fatigue Design Parameters

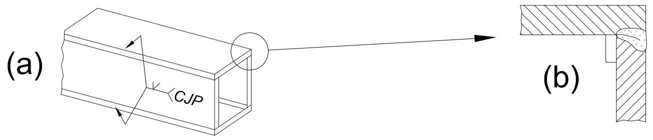
Illustrative Typical Examples

SECTION 3 – WELDED JOINTS JOINING COMPONENTS OF BUILT-UP MEMBERS

3.1

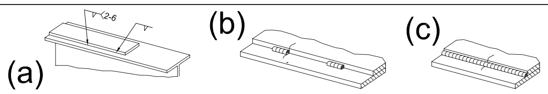


3.2

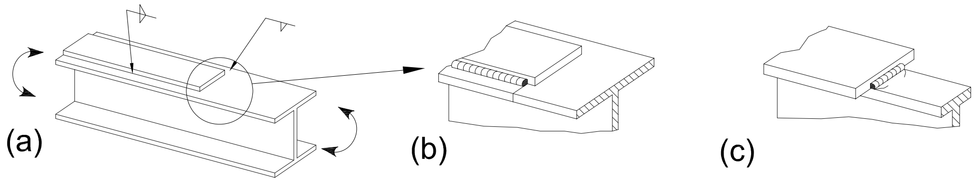


3.3

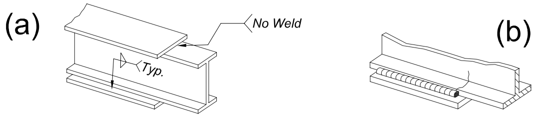
3.4



3.5



3.6



SECTION 4 – LONGITUDINAL FILLET WELDED END CONNECTIONS

4.1

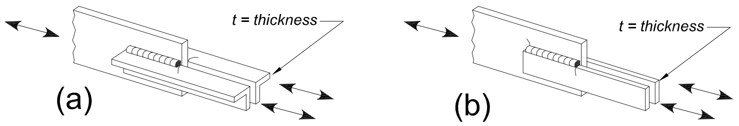


TABLE A-3.1 (cont.)
Fatigue Design Parameters

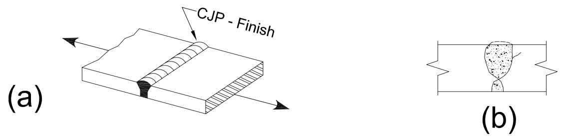
Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS				
5.1 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices in rolled or welded cross sections with welds ground essentially parallel to the direction of stress.	B	120×10^8	16 (110)	From internal discontinuities in filler metal or along the fusion boundary
5.2 Base metal and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in thickness or width made on a slope no greater than 8 to 20%.				From internal discontinuities in filler metal or along fusion boundary or at start of transition when $F_y \geq 90$ ksi (620 MPa)
$F_y < 90$ ksi (620 MPa)	B	120×10^8	16 (110)	
$F_y \geq 90$ ksi (620 MPa)	B′	61×10^8	12 (83)	
5.3 Base metal with F_y equal to or greater than 90 ksi (620 MPa) and weld metal in or adjacent to complete-joint-penetration groove welded splices with welds ground essentially parallel to the direction of stress at transitions in width made on a radius of not less than 2 ft (600 mm) with the point of tangency at the end of the groove weld.	B	120×10^8	16 (110)	From internal discontinuities in filler metal or discontinuities along the fusion boundary
5.4 Base metal and weld metal in or adjacent to the toe of complete-joint-penetration T or corner joints or splices, with or without transitions in thickness having slopes no greater than 8 to 20%, when weld reinforcement is not removed.	C	44×10^8	10 (69)	From surface discontinuity at toe of weld extending into base metal or along fusion boundary.
5.5 Base metal and weld metal at transverse end connections of tension-loaded plate elements using partial-joint-penetration butt or T or corner joints, with reinforcing or contouring fillets, F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at weld root subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^8	10 (69)	
Crack initiating from weld root:	C′	Eqn. A-3-4 or A-3-4M	None provided	

TABLE A-3.1 (cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS

5.1



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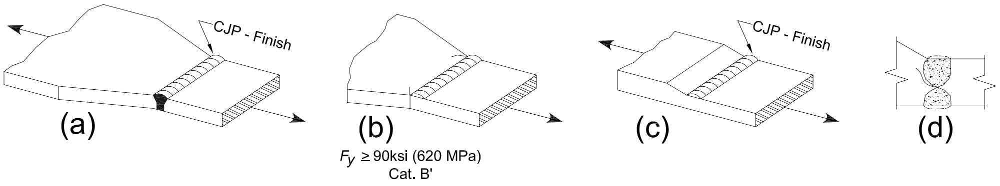
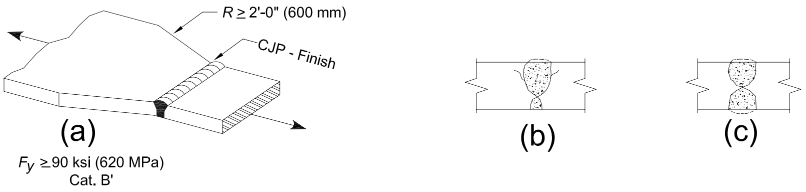
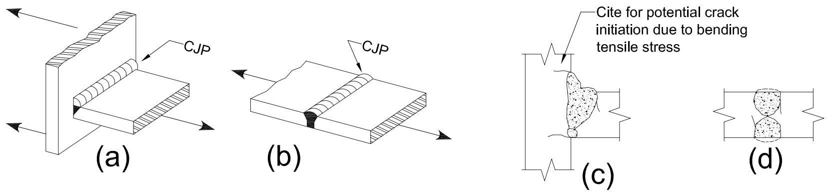


5.3



5.4



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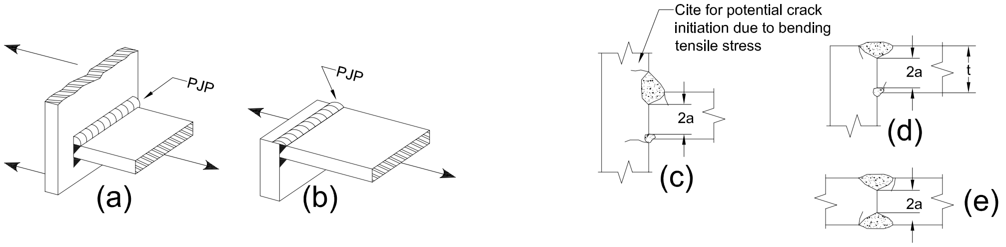


TABLE A-3.1 (cont.)
Fatigue Design Parameters

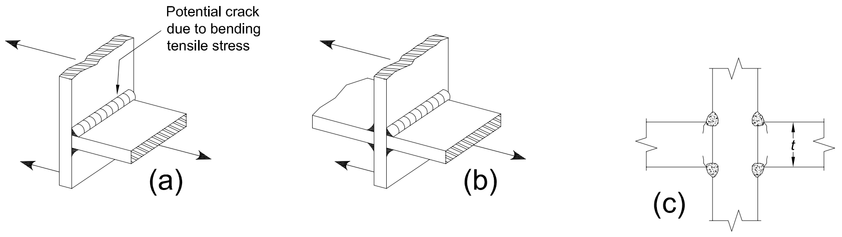
Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)				
5.6 Base metal and filler metal at transverse end connections of tension-loaded plate elements using a pair of fillet welds on opposite sides of the plate. F_{SR} shall be the smaller of the toe crack or root crack stress range.				Initiating from geometrical discontinuity at toe of weld extending into base metal or, initiating at *weld root* subject to tension extending up and then out through weld
Crack initiating from weld toe:	C	44×10^8	10 (69)	
Crack initiating from weld root:	C″	Eqn. A-3-5 or A-3-5M	None provided	
5.7 Base metal of tension loaded plate elements and on girders and rolled beam webs or flanges at toe of transverse fillet welds adjacent to welded transverse stiffeners.	C	44×10^8	10 (69)	From geometrical discontinuity at *toe of fillet* extending into base metal
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS				
6.1 Base metal at details attached by complete-joint-penetration groove welds subject to longitudinal loading only when the detail embodies a transition radius R with the weld termination ground smooth.				Near point of tangency of radius at edge of member
$R \geq$ 24 in. (600 mm)	B	120×10^8	16 (110)	
24 in. $>$ $R \geq$ 6 in. (600 mm $>$ $R \geq$ 150 mm)	C	44×10^8	10 (69)	
6 in. $>$ $R \geq$ 2 in. (150 mm $>$ $R \geq$ 50 mm)	D	22×10^8	7 (48)	
2 in. (50 mm) $>$ R	E	11×10^8	4.5 (31)	

TABLE A-3.1 (cont.)
Fatigue Design Parameters

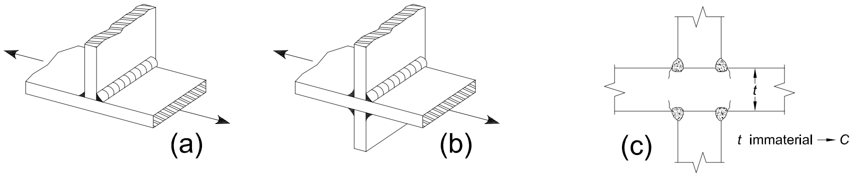
Illustrative Typical Examples

SECTION 5 – WELDED JOINTS TRANSVERSE TO DIRECTION OF STRESS (cont'd)

5.6



5.7



SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS

6.1

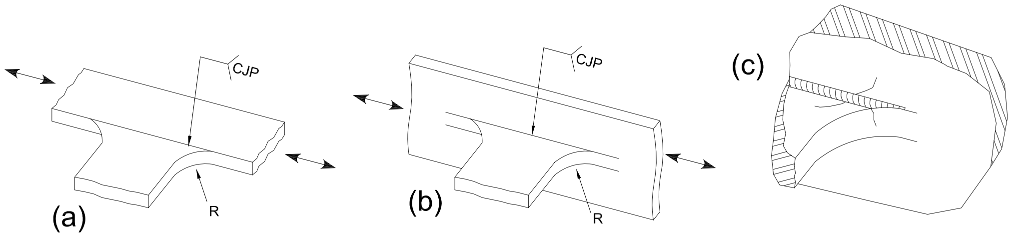


TABLE A-3.1 (cont.)
Fatigue Design Parameters

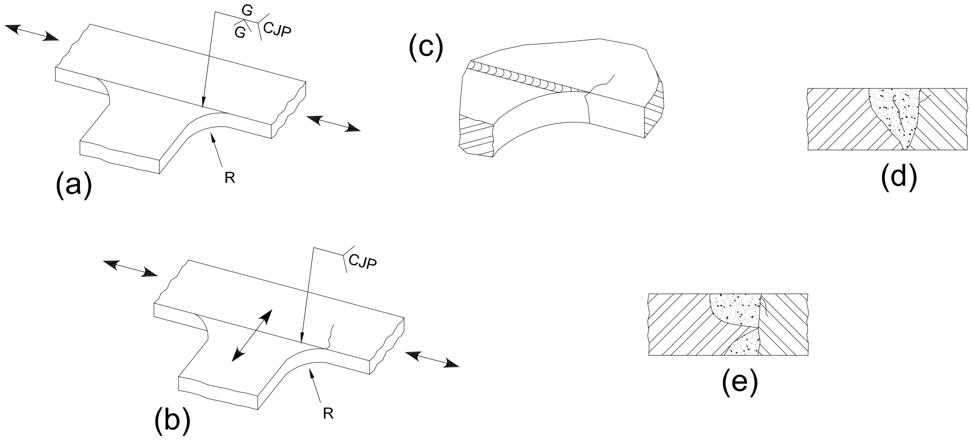
Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.2 Base metal at details of equalthickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth:				
When weld reinforcement is removed:				Near points of tangency of radius or in the weld or at fusion boundary or member or attachment
$R \geq 24$ in. (600 mm)	B	120×10^8	16 (110)	
24 in. $> R \geq 6$ in. (600 mm $> R \geq 150$ mm)	C	44×10^8	10 (69)	
6 in. $> R \geq 2$ in. (150 mm $> R \geq 50$ mm)	D	22×10^8	7 (48)	
2 in. (50 mm) $> R$	E	11×10^8	4.5 (31)	
When weld reinforcement is not removed:				At toe of the weld either along edge of member or the attachment
$R \geq 24$ in. (600 mm)	C	44×10^8	10 (69)	
24 in. $> R \geq 6$ in. (600 mm $> R \geq 150$ mm)	C	44×10^8	10 (69)	
6 in. $> R \geq 2$ in. (150 mm $> R \geq 50$ mm)	D	22×10^8	7 (48)	
2 in. (50 mm) $> R$	E	11×10^8	4.5 (31)	
6.3 Base metal at details of unequal thickness attached by complete-joint-penetration groove welds subject to transverse loading with or without longitudinal loading when the detail embodies a transition radius R with the weld termination ground smooth.				
When weld reinforcement is removed:				
$R > 2$ in. (50 mm)	D	22×10^8	7 (48)	At toe of weld along edge of thinner material
$R \leq 2$ in. (50 mm)	E	11×10^8	4.5 (31)	In weld termination in small radius
When reinforcement is not removed:				
Any radius	E	11×10^8	4.5 (31)	At toe of weld along edge of thinner material

TABLE A-3.1 (cont.)
Fatigue Design Parameters

Illustrative Typical Examples

SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.2



6.3

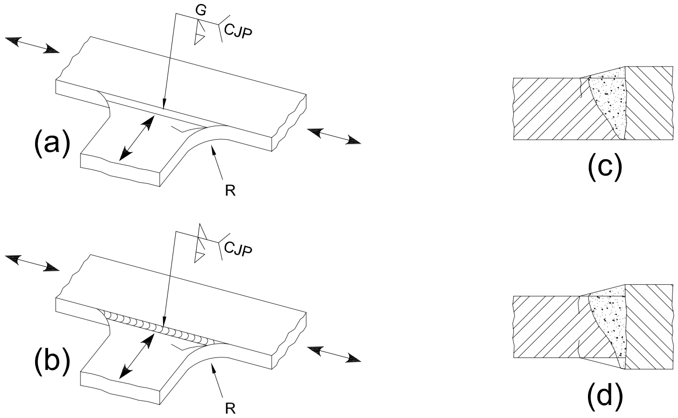


TABLE A-3.1 (cont.)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)				
6.4 Base metal subject to longitudinal stress at transverse members, with or without transverse stress, attached by fillet or partial penetration groove welds parallel to direction of stress when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination or from the toe of the weld extending into member
$R > 2$ in. (50 mm)	D	22×10^8	7 (48)	
$R \leq 2$ in. (50 mm)	E	11×10^8	4.5 (31)	
SECTION 7 – BASE METAL AT SHORT ATTACHMENTS[1]				
7.1 Base metal subject to longitudinal loading at details attached by fillet welds parallel or transverse to the direction of stress where the detail embodies no transition radius and with detail length in direction of stress, *a*, and attachment height normal to the surface of the member, *b*:				In the member at the end of the weld
$a < 2$ in. (50 mm)	C	44×10^8	10 (69)	
2 in. (50 mm) $\leq a \leq 12\,b$ or 4 in (100 mm)	D	22×10^8	7 (48)	
$a > 12b$ or 4 in. (100 mm) when b is ≤ 1 in. (25 mm)	E	11×10^8	4.5 (31)	
$a > 12b$ or 4 in. (100 mm) when b is > 1 in. (25 mm)	E′	3.9×10^8	2.6 (18)	
7.2 Base metal subject to longitudinal *stress* at details attached by fillet or partial-joint-penetration groove welds, with or without transverse *load* on detail, when the detail embodies a transition radius, *R*, with weld termination ground smooth:				In weld termination extending into member
$R > 2$ in. (50 mm)	D	22×10^8	7 (48)	
$R \leq 2$ in. (50 mm)	E	11×10^8	4.5 (31)	

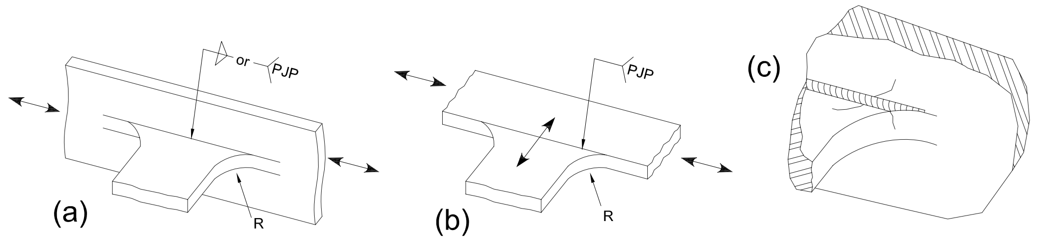
[1] "Attachment" as used herein, is defined as any steel detail welded to a member which, by its mere presence and independent of its loading, causes a discontinuity in the *stress* flow in the member and thus reduces the *fatigue* resistance.

TABLE A-3.1 (cont.)
Fatigue Design Parameters

Illustrative Typical Examples

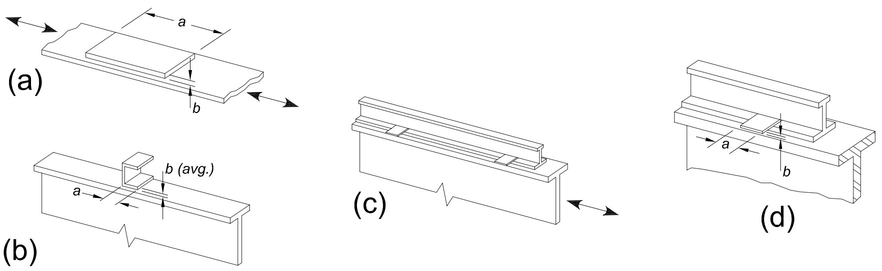
SECTION 6 – BASE METAL AT WELDED TRANSVERSE MEMBER CONNECTIONS (cont'd)

6.4



SECTION 7 – BASE METAL AT SHORT ATTACHMENTS

7.1



7.2

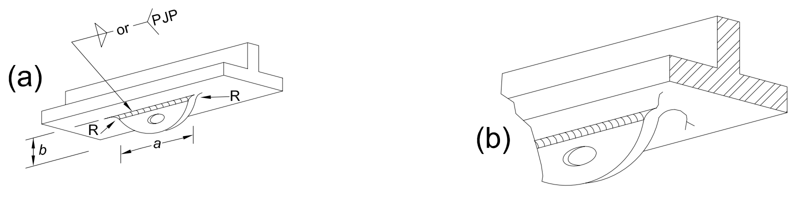
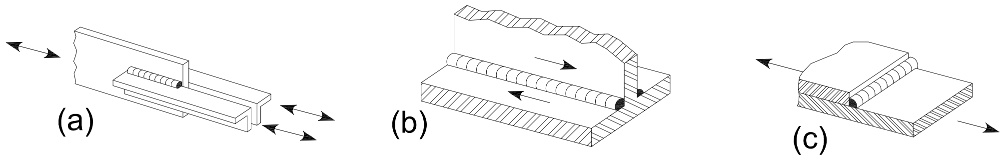
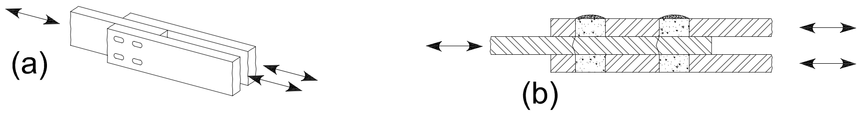
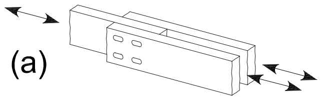
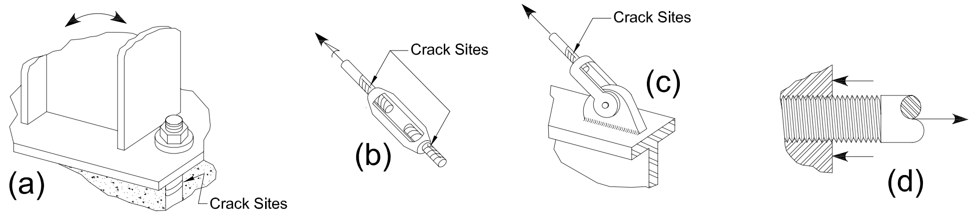


TABLE A-3.1 (cont.)
Fatigue Design Parameters

Description	Stress Category	Constant C_f	Threshold F_{TH} ksi (MPa)	Potential Crack Initiation Point
SECTION 8 - MISCELLANEOUS				
8.1 Base metal at stud-type shear connectors attached by fillet or electric stud welding.	C	44×10^8	10 (69)	At toe of weld in base metal
8.2 Shear on throat of continuous or intermittent longitudinal or transverse fillet welds.	F	150×10^{10} (Eqn. A-3-2 or A-3-2M)	8 (55)	In throat of weld
8.3 Base metal at plug or slot welds.	E	11×10^8	4.5 (31)	At end of weld in base metal
8.4 Shear on plug or slot welds.	F	150×10^{10} (Eqn. A-3-2 or A-3-2M)	8 (55)	At *faying surface*
8.5 Not fully tightened high-strength bolts, common bolts, threaded anchor rods and hanger rods with cut, ground or rolled threads. Stress range on tensile stress area due to live load plus prying action when applicable.	E′	3.9×10^8	7 (48)	At the root of the threads extending into the tensile *stress* area

TABLE A-3.1 (cont.)
Fatigue Design Parameters

	Illustrative Typical Examples
SECTION 8 – MISCELLANEOUS	
8.1	
8.2	
8.3	
8.4	
8.5	

APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

This appendix provides criteria for the design and evaluation of *structural steel* components, systems and frames for fire conditions. These criteria provide for the determination of the heat input, thermal expansion and degradation in mechanical properties of materials at elevated temperatures that cause progressive decrease in strength and *stiffness* of structural components and systems at elevated temperatures.

The appendix is organized as follows:

4.1. General Provisions
4.2. Structural Design for Fire Conditions by Analysis
4.3. Design by Qualification Testing

4.1. GENERAL PROVISIONS

The methods contained in this appendix provide regulatory evidence of compliance in accordance with the design applications outlined in this section.

The appendix uses the following terms in addition to the terms in the Glossary.

Active fire protection: Building materials and systems that are activated by a fire to mitigate adverse effects or to notify people to take some action mitigate adverse effects.

Compartmentation: The enclosure of a building space with elements that have a specific fire endurance.

Convective heat transfer: The transfer of thermal energy from a point of higher temperature to a point of lower temperature through the motion of an intervening medium.

Design-basis fire: A set of conditions that define the development of a fire and the spread of combustion products throughout a building or portion thereof.

Elevated temperatures: Heating conditions experienced by building elements or structures as a result of fire, which are in excess of the anticipated ambient conditions.

Fire: Destructive burning, as manifested by any or all of the following: light, flame, heat, or smoke.

Fire barrier: Element of construction formed of fire-resisting materials and tested in accordance with ASTM Standard E119, or other approved standard fire resistance test, to demonstrate compliance with the Building Code.

Fire endurance: A measure of the elapsed time during which a material or assembly continues to exhibit fire resistance.

Fire resistance: That property of assemblies that prevents or retards the passage of excessive heat, hot gases or flames under conditions of use and enables them to continue to perform a stipulated function.

Fire resistance rating: The period of time a building element, component or assembly maintains the ability to contain a fire, continues to perform a given structural function, or both, as determined by test or methods based on tests.

Flashover: The rapid transition to a state of total surface involvement in a fire of combustible materials within an enclosure.

Heat flux: Radiant energy per unit surface area.

Heat release rate: The rate at which thermal energy is generated by a burning material.

Passive fire protection: Building materials and systems whose ability to resist the effects of fire does not rely on any outside activating condition or mechanism.

Performance-based design: An engineering approach to structural design that is based on agreed-upon performance goals and objectives, engineering analysis and quantitative assessment of alternatives against those design goals and objectives using accepted engineering tools, methodologies and performance criteria.

Prescriptive design: A design method that documents compliance with general criteria established in a building code.

Restrained construction: Floor and roof assemblies and individual *beams* in buildings where the surrounding or supporting structure is capable of resisting substantial thermal expansion throughout the range of anticipated elevated temperatures.

Unrestrained construction: Floor and roof assemblies and individual *beams* in buildings that are assumed to be free to rotate and expand throughout the range of anticipated elevated temperatures.

4.1.1. Performance Objective

Structural components, members and building frame systems shall be designed so as to maintain their load-bearing function during the design-basis fire and to satisfy other performance requirements specified for the building occupancy.

Deformation criteria shall be applied where the means of providing structural fire resistance, or the design criteria for fire barriers, requires consideration of the deformation of the load-carrying structure.

Within the compartment of fire origin, *forces* and deformations from the design-basis fire shall not cause a breach of horizontal or vertical compartmentation.

4.1.2. Design by Engineering Analysis

The analysis methods in Section 4.2 are permitted to be used to document the anticipated performance of steel framing when subjected to design-basis fire scenarios. Methods in Section 4.2 provide evidence of compliance with performance objectives established in Section 4.1.1.

The analysis methods in Section 4.2 are permitted to be used to demonstrate an equivalency for an alternative material or method, as permitted by the building code.

4.1.3. Design by Qualification Testing

The qualification testing methods in Section 4.3 are permitted to be used to document the fire resistance of steel framing subject to the standardized fire testing protocols required by building codes.

4.1.4. Load Combinations and Required Strength

The *required strength* of the structure and its elements shall be determined from the following *gravity load combination*:

$$[0.9 \text{ or } 1.2]D + T + 0.5L + 0.2S \qquad \text{(A-4-1)}$$

where

D = *nominal dead load*
L = nominal occupancy live load
S = nominal snow load
T = nominal *forces* and deformations due to the design-basis fire defined in Section 4.2.1

A lateral *notional load*, $N_i = 0.002Y_i$, as defined in Appendix 7.2, where N_i = notional *lateral load* applied at framing level i and Y_i = *gravity load* from combination A-4-1 acting on framing level i, shall be applied in combination with the loads stipulated in Equation A-4-1. Unless otherwise stipulated by the *authority having jurisdiction*, D, L and S shall be the *nominal loads* specified in ASCE 7.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

It is permitted to design structural members, components and building frames for elevated temperatures in accordance with the requirements of this section.

4.2.1. Design-Basis Fire

A design-basis fire shall be identified to describe the heating conditions for the structure. These heating conditions shall relate to the fuel commodities and compartment characteristics present in the assumed fire area. The fuel *load* density based on the occupancy of the space shall be considered when determining the total fuel *load*. Heating conditions shall be specified either in terms of a heat flux or temperature of the upper gas layer created by the fire. The variation of the heating conditions with time shall be determined for the duration of the fire.

When the analysis methods in Section 4.2 are used to demonstrate an equivalency as an alternative material or method as permitted by a building code, the design-basis fire shall be determined in accordance with ASTM E119.

4.2.1.1. Localized Fire

Where the heat release rate from the fire is insufficient to cause flashover, a localized fire exposure shall be assumed. In such cases, the fuel composition, arrangement of the fuel array and floor area occupied by the fuel shall be used to determine the radiant heat flux from the flame and smoke plume to the structure.

4.2.1.2. Post-Flashover Compartment Fires

Where the heat release rate from the fire is sufficient to cause flashover, a post-flashover compartment fire shall be assumed. The determination of the temperature versus time profile resulting from the fire shall include fuel *load*, ventilation characteristics to the space (natural and mechanical), compartment dimensions and thermal characteristics of the compartment boundary.

4.2.1.3. Exterior Fires

The exposure of exterior structure to flames projecting from windows or other wall openings as a result of a post-flashover compartment fire shall be considered along with the radiation from the interior fire through the opening. The shape and length of the flame projection shall be used along with the distance between the flame and the exterior steelwork to determine the heat flux to the steel. The method identified in Section 4.2.1.2 shall be used for describing the characteristics of the interior compartment fire.

4.2.1.4. Fire Duration

The fire duration in a particular area shall be determined by considering the total combustible mass, in other words, fuel *load* available in the space. In the case of either a localized fire or a post-flashover compartment fire, the time duration shall be determined as the total combustible mass divided by the mass loss rate, except where determined from Section 4.2.1.2.

4.2.1.5. Active Fire Protection Systems

The effects of active fire protection systems shall be considered when describing the design-basis fire.

Where automatic smoke and heat vents are installed in nonsprinklered spaces, the resulting smoke temperature shall be determined from calculation.

4.2.2. Temperatures in Structural Systems under Fire Conditions

Temperatures within structural members, components and frames due to the heating conditions posed by the design-basis fire shall be determined by a heat transfer analysis.

Table A-4.2.1
Properties of Steel at Elevated Temperatures

Steel Tempreature (°F)[°C]	$k_E = E_m / E$	$k_y = F_{ym} / F_y$	$k_u = F_{um} / F_y$
68 [20]	*	*	*
200 [93]	1.00	*	*
400 [204]	0.90	*	*
600 [316]	0.78	*	*
750 [399]	0.70	1.00	1.00
800 [427]	0.67	0.94	0.94
1000 [538]	0.49	0.66	0.66
1200 [649]	0.22	0.35	0.35
1400 [760]	0.11	0.16	0.16
1600 [871]	0.07	0.07	0.07
1800 [982]	0.05	0.04	0.04
2000 [1093]	0.02	0.02	0.02
2200 [1204]	0.00	0.00	0.00

*Use ambient properties.

4.2.3. Material Strengths at Elevated Temperatures

Material properties at elevated temperatures shall be determined from test data. In the absence of such data, it is permitted to use the material properties stipulated in this section. These relationships do not apply for steels with a *yield strength* in excess of 65 ksi (448 MPa) or concretes with specified compression strength in excess of 8,000 psi (55 MPa).

4.2.3.1. Thermal Elongation

Thermal expansion of structural and reinforcing steels: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 7.8×10^{-6}/°F (1.4×10^{-5}/°C).

Thermal expansion of normal weight concrete: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 1.0×10^{-5}/°F (1.8×10^{-5}/°C).

Thermal expansion of lightweight concrete: For calculations at temperatures above 150 °F (65 °C), the coefficient of thermal expansion shall be 4.4×10^{-6}/°F (7.9×10^{-6}/°C).

4.2.3.2. Mechanical Properties at Elevated Temperatures

The deterioration in strength and *stiffness* of structural members, components, and systems shall be taken into account in the *structural analysis* of the frame. The values F_{ym}, F_{um}, E_m, f'_{cm}, E_{cm} and ε_{cu} at elevated temperature to be used in *structural analysis*, expressed as the ratio with respect to the property at ambient, assumed to be 68 °F (20 °C), shall be defined as in Tables A-4.2.1 and A-4.2.2. It is permitted to interpolate between these values.

Table A-4.2.2

Properties of Concrete at Elevated Temperatures

Concrete Temperature (°F)[°C]	$k_c = f'_{cm}/f'_c$			$\varepsilon_{cu}(\%)$
	NWC	LWC	E_{cm}/E_c	LWC
68 [20]	1.00	1.00	1.00	0.25
200 [93]	0.95	1.00	0.93	0.34
400 [204]	0.90	1.00	0.75	0.46
550 [288]	0.86	1.00	0.61	0.58
600 [316]	0.83	0.98	0.57	0.62
800 [427]	0.71	0.85	0.38	0.80
1000 [538]	0.54	0.71	0.20	1.06
1200 [649]	0.38	0.58	0.092	1.32
1400 [760]	0.21	0.45	0.073	1.43
1600 [871]	0.10	0.31	0.055	1.49
1800 [982]	0.05	0.18	0.036	1.50
2000 [1093]	0.01	0.05	0.018	1.50
2200 [1204]	0.00	0.00	0.00	–

For lightweight concrete (LWC), values of ε_{cu} shall be obtained from tests.

4.2.4. Structural Design Requirements

4.2.4.1. General Structural Integrity

The structural frame shall be capable of providing adequate strength and deformation capacity to withstand, as a system, the structural actions developed during the fire within the prescribed limits of deformation. The *structural system* shall be designed to sustain local damage with the structural system as a whole remaining stable.

Continuous *load* paths shall be provided to transfer all *forces* from the exposed region to the final point of resistance. The foundation shall be designed to resist the forces and to accommodate the deformations developed during the design-basis fire.

4.2.4.2. Strength Requirements and Deformation Limits

Conformance of the structural system to these requirements shall be demonstrated by constructing a mathematical model of the structure based on principles of structural mechanics and evaluating this model for the internal forces and deformations in the members of the structure developed by the temperatures from the design-basis fire.

Individual members shall be provided with adequate strength to resist the shears, axial forces and moments determined in accordance with these provisions.

Connections shall develop the strength of the connected members or the forces indicated above. Where the means of providing fire resistance requires the consideration of deformation criteria, the deformation of the structural system, or members thereof, under the design-basis fire shall not exceed the prescribed limits.

4.2.4.3. Methods of Analysis

4.2.4.3a. Advanced Methods of Analysis

The methods of analysis in this section are permitted for the design of all steel building structures for fire conditions. The design-basis fire exposure shall be that determined in Section 4.2.1. The analysis shall include both a thermal response and the mechanical response to the design-basis fire.

The *thermal response* shall produce a temperature field in each structural element as a result of the design-basis fire and shall incorporate temperature-dependent thermal properties of the structural elements and fire-resistive materials as per Section 4.2.2.

The*mechanical response* results in forces and deflections in the structural system subjected to the thermal response calculated from the design-basis fire. The mechanical response shall take into account explicitly the deterioration in strength and *stiffness* with increasing temperature, the effects of thermal expansions and large deformations. Boundary conditions and connection fixity must represent the proposed structural design. Material properties shall be defined as per Section 4.2.3.

The resulting analysis shall consider all relevant *limit states*, such as excessive deflections, connection fractures, and overall or *local buckling*.

4.2.4.3b. Simple Methods of Analysis

The methods of analysis in this section are applicable for the evaluation of the performance of individual members at elevated temperatures during exposure to fire.

The support and restraint conditions (forces, moments and boundary conditions) applicable at normal temperatures may be assumed to remain unchanged throughout the fire exposure.

(1) Tension members

It is permitted to model the thermal response of a tension element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 4.2.1.

The *design strength* of a tension member shall be determined using the provisions of Chapter D, with steel properties as stipulated in Section 4.2.3 and assuming a uniform temperature over the cross section using the temperature equal to the maximum steel temperature.

(2) Compression members

It is permitted to model the thermal response of a compression element using a one-dimensional heat transfer equation with heat input as directed by the design-basis fire defined in Section 4.2.1.

The design strength of a compression member shall be determined using the provisions of Chapter E with steel properties as stipulated in Section 4.2.3.

(3) Flexural members

It is permitted to model the thermal response of flexural elements using a one-dimensional heat transfer equation to calculate bottom flange temperature and to assume that this bottom flange temperature is constant over the depth of the member.

The design strength of a flexural member shall be determined using the provisions of Chapter F with steel properties as stipulated in Section 4.2.3.

(4) Composite floor members

It is permitted to model the thermal response of flexural elements supporting a concrete slab using a one-dimensional heat transfer equation to calculate bottom flange temperature. That temperature shall be taken as constant between the bottom flange and mid-depth of the web and shall decrease linearly by no more than 25 percent from the mid-depth of the web to the top flange of the *beam*.

The design strength of a *composite* flexural member shall be determined using the provisions of Chapter I, with reduced *yield stresses* in the steel consistent with the temperature variation described under thermal response.

4.2.4.4. Design Strength

The design strength shall be determined as in Section B3.3. The *nominal strength*, R_n, shall be calculated using material properties, as stipulated in Section 4.2.3, at the temperature developed by the design-basis fire.

4.3. DESIGN BY QUALIFICATION TESTING

4.3.1. Qualification Standards

Structural members and components in steel buildings shall be qualified for the rating period in conformance with ASTM E119. It shall be permitted to demonstrate compliance with these requirements using the procedures specified for steel construction in Section 5 of ASCE/SFPE 29.

4.3.2. Restrained Construction

For floor and roof assemblies and individual *beams* in buildings, a restrained condition exists when the surrounding or supporting structure is capable of resisting actions caused by thermal expansion throughout the range of anticipated elevated temperatures.

Steel beams, girders and frames supporting concrete slabs that are welded or bolted to integral framing members (in other words, *columns*, girders) shall be considered restrained construction.

4.3.3. Unrestrained Construction

Steel beams, girders and frames that do not support a concrete slab shall be considered unrestrained unless the members are bolted or welded to surrounding construction that has been specifically designed and detailed to resist actions caused by thermal expansion.

A steel member bearing on a wall in a single span or at the end span of multiple spans shall be considered unrestrained unless the wall has been designed and detailed to resist effects of thermal expansion.

APPENDIX 5

EVALUATION OF EXISTING STRUCTURES

This appendix applies to the evaluation of the strength and *stiffness* under static vertical (gravity) *loads* of existing structures by *structural analysis*, by *load* tests, or by a combination of *structural analysis* and *load* tests when specified by the engineer of record or in the contract documents. For such evaluation, the steel grades are not limited to those listed in Section A3.1. This appendix does not address *load* testing for the effects of seismic *loads* or moving *loads* (vibrations).

The Appendix is organized as follows:

5.1. General Provisions
5.2. Material Properties
5.3. Evaluation by Structural Analysis
5.4. Evaluation by Load Tests
5.5. Evaluation Report

5.1. GENERAL PROVISIONS

These provisions shall be applicable when the evaluation of an existing steel structure is specified for (a) verification of a specific set of design loadings or (b) determination of the *available strength* of a *load* resisting member or system. The evaluation shall be performed by *structural analysis* (Section 5.3), by *load* tests (Section 5.4), or by a combination of *structural analysis* and load tests, as specified in the contract documents. Where load tests are used, the *engineer of record* shall first analyze the structure, prepare a testing plan, and develop a written procedure to prevent excessive permanent deformation or catastrophic collapse during testing.

5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

The *engineer of record* shall determine the specific tests that are required from Section 5.2.2 through 5.2.6 and specify the locations where they are required. Where available, the use of applicable project records shall be permitted to reduce or eliminate the need for testing.

2. Tensile Properties

Tensile properties of members shall be considered in evaluation by *structural analysis* (Section 5.3) or *load* tests (Section 5.4). Such properties shall include the *yield stress*, *tensile strength* and *percent elongation*. Where available, certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM A6/A6M or A568/A568M, as applicable, shall be permitted for this purpose. Otherwise, tensile tests shall be conducted in accordance with ASTM A370 from samples cut from components of the structure.

3. Chemical Composition

Where welding is anticipated for repair or modification of existing structures, the chemical composition of the steel shall be determined for use in preparing a welding procedure specification (WPS). Where available, results from certified mill test reports or certified reports of tests made by the fabricator or a testing laboratory in accordance with ASTM procedures shall be permitted for this purpose. Otherwise, analyses shall be conducted in accordance with ASTM A751 from the samples used to determine tensile properties, or from samples taken from the same locations.

4. Base Metal Notch Toughness

Where welded tension splices in heavy shapes and plates as defined in Section A3.1d are critical to the performance of the structure, the *Charpy V-Notch toughness s*hall be determined in accordance with the provisions of Section A3.1d. If the notch toughness so determined does not meet the provisions of Section A3.1d, the *engineer of record* shall determine if remedial actions are required.

5. Weld Metal

Where structural performance is dependent on existing welded *connections*, representative samples of *weld metal* shall be obtained. Chemical analysis and mechanical tests shall be made to characterize the weld metal. A determination shall be made of the magnitude and consequences of imperfections. If the requirements of AWS D1.1 are not met, the *engineer of record* shall determine if remedial actions are required.

6. Bolts and Rivets

Representative samples of bolts shall be inspected to determine markings and classifications. Where bolts cannot be properly identified visually, representative samples shall be removed and tested to determine *tensile strength* in accordance with ASTM F606 or ASTM F606M and the bolt classified accordingly. Alternatively, the assumption that the bolts are ASTM A307 shall be permitted. Rivets shall be assumed to be ASTM A502, Grade 1, unless a higher grade is established through documentation or testing.

5.3. EVALUATION BY STRUCTURAL ANALYSIS

1. Dimensional Data

All dimensions used in the evaluation, such as spans, *column* heights, member spacings, bracing locations, cross section dimensions, thicknesses and *connection* details, shall be determined from a field survey. Alternatively, when available, it shall be permitted to determine such dimensions from applicable project design or shop drawings with field verification of critical values.

2. Strength Evaluation

Forces (*load effects*) in members and connections shall be determined by *structural analysis* applicable to the type of structure evaluated. The load effects shall be determined for the *loads* and *factored load combinations* stipulated in Section B2.

The *available strength* of members and connections shall be determined from applicable provisions of Chapters B through K of this Specification.

3. Serviceability Evaluation

Where required, the deformations at *service loads* shall be calculated and reported.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Load Rating by Testing

To determine the *load* rating of an existing floor or roof structure by testing, a test load shall be applied incrementally in accordance with the *engineer of record's* plan. The structure shall be visually inspected for signs of distress or imminent failure at each load level. Appropriate measures shall be taken if these or any other unusual conditions are encountered.

The tested strength of the structure shall be taken as the maximum applied test load plus the in-situ dead load. The live load rating of a floor structure shall be determined by setting the tested strength equal to $1.2D + 1.6L$, where D is the nominal dead load and L is the nominal live load rating for the structure. The nominal live load rating of the floor structure shall not exceed that which can be calculated using applicable provisions of the specification. For roof structures, L_r, S, or R as defined in the Symbols, shall be substituted for L. More severe *load combinations* shall be used where required by *applicable building codes*.

Periodic unloading shall be considered once the *service load* level is attained and after the onset of inelastic structural behavior is identified to document the amount of permanent set and the magnitude of the inelastic deformations. Deformations of the structure, such as member deflections, shall be monitored at critical locations during the test, referenced to the initial position before loading. It shall be demonstrated, while maintaining maximum test load for one hour that the deformation of the structure does not increase by more than 10 percent above that at the beginning of the holding period. It is permissible to repeat the sequence if necessary to demonstrate compliance.

Deformations of the structure shall also be recorded 24 hours after the test loading is removed to determine the amount of permanent set. Because the amount of acceptable permanent deformation depends on the specific structure, no limit is specified for permanent deformation at maximum loading. Where it is not feasible to load test the entire structure, a segment or zone of not less than one complete bay, representative of the most critical conditions, shall be selected.

2. Serviceability Evaluation

When load tests are prescribed, the structure shall be loaded incrementally to the service load level. Deformations shall be monitored for a period of one hour. The structure shall then be unloaded and the deformation recorded.

5.5. EVALUATION REPORT

After the evaluation of an existing structure has been completed, the *engineer of record* shall prepare a report documenting the evaluation. The report shall indicate whether the evaluation was performed by *structural analysis*, by *load* testing or by a combination of structural analysis and load testing. Furthermore, when testing is performed, the report shall include the loads and load combination used and the load-deformation and time-deformation relationships observed. All relevant information obtained from design drawings, mill test reports and auxiliary material testing shall also be reported. Finally, the report shall indicate whether the structure, including all members and *connections*, is adequate to withstand the *load effects*.

APPENDIX 6

STABILITY BRACING FOR COLUMNS AND BEAMS

This appendix addresses the minimum brace strength and *stiffness* necessary to provide member *strengths* based on the *unbraced length* between braces with an *effective length factor*, K, equal to 1.0.

The appendix is organized as follows:

6.1. General Provisions
6.2. Columns
6.3. Beams

User Note: The requirements for the stability of braced-frame systems are provided in Chapter C. The provisions in this appendix apply to bracing, intended to stabilize individual members.

6.1. GENERAL PROVISIONS

Bracing is assumed to be perpendicular to the members to be braced; for inclined or *diagonal bracing*, the brace strength (*force* or moment) and *stiffness* (force per unit displacement or moment per unit rotation) shall be adjusted for the angle of inclination. The evaluation of the stiffness furnished by a brace shall include its member and geometric properties, as well as the effects of *connections* and anchoring details.

Two general types of bracing systems are considered, relative and nodal. A *relative brace* controls the movement of the brace point with respect to adjacent braced points. A *nodal brace* controls the movement at the braced point without direct interaction with adjacent braced points. The *available strength* and stiffness of the bracing shall equal or exceed the required limits unless analysis indicates that smaller values are justified by analysis.

A *second-order analysis* that includes an initial out-of-straightness of the member to obtain brace strength and stiffness is permitted in lieu of the requirements of this appendix.

6.2. COLUMNS

It is permitted to brace an individual *column* at end and intermediate points along its length by either relative or nodal bracing systems. It is assumed that *nodal braces* are equally spaced along the column.

1. Relative Bracing

The required brace strength is

$$P_{br} = 0.004P_r \tag{A-6-1}$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{2P_r}{L_b}\right) \text{(LRFD)} \qquad \beta_{br} = \Omega\left(\frac{2P_r}{L_b}\right) \text{(ASD)} \tag{A-6-2}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

L_b = distance between braces, in. (mm)

For design according to Section B3.3 (LRFD)

P_r = *required axial compressive strength* using *LRFD load combinations*, kips (N)

For design according to Section B3.4 (ASD)

P_r = required axial compressive strength using *ASD load combinations*, kips (N)

2. Nodal Bracing

The required brace strength is

$$P_{br} = 0.01P_r \tag{A-6-3}$$

The required brace stiffness is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{8P_r}{L_b}\right) \text{(LRFD)} \qquad \beta_{br} = \Omega\left(\frac{8P_r}{L_b}\right) \text{(ASD)} \tag{A-6-4}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

For design according to Section B3.3 (LRFD)

P_r = *required axial compressive strength* using *LRFD load combinations*, kips (N)

For design according to Section B3.4 (ASD)

P_r = required axial compressive strength using *ASD load combinations*, kips (N)

When L_b is less than L_q, where L_q is the maximum *unbraced length* for the required *column force* with K equal to 1.0, then L_b in Equation A-6-4 is permitted to be taken equal to L_q.

6.3. BEAMS

At points of support for *beams*, girders and trusses, restraint against rotation about their longitudinal axis shall be provided. Beam bracing shall prevent the relative displacement of the top and bottom flanges, in other words, twist of the section. Lateral *stability* of beams shall be provided by *lateral bracing*, *torsional bracing* or a combination of the two. In members subjected to *double curvature* bending, the inflection point shall not be considered a brace point.

1. Lateral Bracing

Bracing shall be attached near the compression flange, except for a cantilevered member, where an end brace shall be attached near the top (tension) flange. Lateral bracing shall be attached to both flanges at the brace point nearest the inflection point for beams subjected to double curvature bending along the length to be braced.

1a. Relative Bracing

The required brace strength is

$$P_{br} = 0.008 M_r C_d / h_o \qquad \text{(A-6-5)}$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{4 M_r C_d}{L_b h_o}\right) \text{(LRFD)} \qquad \beta_{br} = \Omega\left(\frac{4 M_r C_d}{L_b h_o}\right) \text{(ASD)} \qquad \text{(A-6-6)}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

h_o = distance between flange centroids, in. (mm)
C_d = 1.0 for bending in *single curvature*; 2.0 for double curvature; $C_d = 2.0$ only applies to the brace closest to the inflection point
L_b = laterally *unbraced length*, in. (mm)

For design according to Section B3.3 (LRFD)

M_r = *required flexural strength* using *LRFD load combinations*, kip-in. (N-mm)

For design according to Section B3.4 (ASD)

M_r = required flexural strength using *ASD load combinations*, kip-in. (N-mm)

1b. Nodal Bracing

The required brace strength is

$$P_{br} = 0.02 M_r C_d / h_o \qquad \text{(A-6-7)}$$

The required brace *stiffness* is

$$\beta_{br} = \frac{1}{\phi}\left(\frac{10 M_r C_d}{L_b h_o}\right) \text{(LRFD)} \qquad \beta_{br} = \Omega\left(\frac{10 M_r C_d}{L_b h_o}\right) \text{(ASD)} \qquad \text{(A-6-8)}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

For design according to Section B3.3 (LRFD)

M_r = required flexural strength using LRFD load combinations, kip-in. (N-mm)

For design according to Section B3.4 (ASD)

M_r = required flexural strength using ASD load combinations, kip-in. (N-mm)

When L_b is less than L_q, the maximum unbraced length for M_r, then L_b in Equation A-6-8 shall be permitted to be taken equal to L_q.

2. Torsional Bracing

It is permitted to provide either nodal or continuous *torsional bracing* along the *beam* length. It is permitted to attach the bracing at any cross-sectional location and it need not be attached near the compression flange. The *connection* between a torsional brace and the beam shall be able to support the required moment given below.

2a. Nodal Bracing

The required bracing moment is

$$M_{br} = \frac{0.024 M_r L}{n C_b L_b} \tag{A-6-9}$$

The required cross-frame or *diaphragm* bracing *stiffness* is

$$\beta_{Tb} = \frac{\beta_T}{\left(1 - \frac{\beta_T}{\beta_{\text{sec}}}\right)} \tag{A-6-10}$$

where

$$\beta_T = \frac{1}{\phi}\left(\frac{2.4 L M_r^2}{n E I_y C_b^2}\right) \text{(LRFD)} \qquad \beta_T = \Omega\left(\frac{2.4 L M_r^2}{n E I_y C_b^2}\right) \text{(ASD)} \tag{A-6-11}$$

$$\beta_{\text{sec}} = \frac{3.3E}{h_o}\left(\frac{1.5 h_o t_w^3}{12} + \frac{t_s b_s^3}{12}\right) \tag{A-6-12}$$

where

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 3.00 \text{ (ASD)}$$

User Note: $\Omega = 1.5^2/\phi = 3.00$ in Equation A-6-11 because the moment term is squared.

L = span length, in. (mm)
n = number of *nodal braced* points within the span
E = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)

I_y = out-of-plane moment of inertia, in.^{4} (mm^4)
C_b = modification factor defined in Chapter F
t_w = *beam* web thickness, in. (mm)
t_s = web *stiffener* thickness, in. (mm)
b_s = stiffener width for one-sided stiffeners (use twice the individual stiffener width for pairs of stiffeners), in. (mm)
β_T = brace *stiffness* excluding web distortion, kip-in./radian (N-mm/radian)
β_{sec} = web *distortional stiffness*, including the effect of web *transverse stiffeners*, if any, kip-in./radian (N-mm/radian)

For design according to Section B3.3 (LRFD)

M_r = *required flexural strength* using *LRFD load combinations*, kip-in. (N-mm)

For design according to Section B3.4 (ASD)

M_r = required flexural strength using *ASD load combinations*, kip-in. (N-mm)

If $\beta_{sec} < \beta_T$, Equation A-6-10 is negative, which indicates that torsional *beam* bracing will not be effective due to inadequate web distortional stiffness.

When required, the web stiffener shall extend the full depth of the braced member and shall be attached to the flange if the torsional brace is also attached to the flange. Alternatively, it shall be permissible to stop the stiffener short by a distance equal to $4t_w$ from any *beam* flange that is not directly attached to the torsional brace. When L_b is less than L_q, then L_b in Equation A-6-9 shall be permitted to be taken equal to L_q.

2b. Continuous Torsional Bracing

For continuous bracing, use Equations A-6-9, A-6-10 and A-6-13 with *L*/*n* taken as 1.0 and L_b taken as L_q; the bracing moment and stiffness are given per unit span length. The distortional stiffness for an unstiffened web is

$$\beta_{sec} = \frac{3.3Et_w^3}{12h_o} \tag{A-6-13}$$

APPENDIX 7

DIRECT ANALYSIS METHOD

This appendix addresses the *direct analysis method* for structural systems comprised of *moment frames*, *braced frames*, *shear walls*, or combinations thereof.

The appendix is organized as follows:

7.1. General Requirements
7.2. Notional Loads
7.3. Design-Analysis Constraints

7.1. GENERAL REQUIREMENTS

Members shall satisfy the provisions of Section H1 with the nominal *column* strengths, P_n, determined using $K = 1.0$. The *required strengths* for members, *connections* and other structural elements shall be determined using a second-order *elastic analysis* with the constraints presented in Section 7.3. All component and *connection* deformations that contribute to the lateral displacement of the structure shall be considered in the analysis.

7.2. NOTIONAL LOADS

Notional loads shall be applied to the lateral framing system to account for the effects of geometric imperfections, inelasticity, or both. *Notional loads* are *lateral loads* that are applied at each framing level and specified in terms of the *gravity loads* applied at that level. The *gravity load* used to determine the *notional load* shall be equal to or greater than the *gravity load* associated with the *load combination* being evaluated. *Notional loads* shall be applied in the direction that adds to the destabilizing effects under the specified *load combination*.

7.3. DESIGN-ANALYSIS CONSTRAINTS

(1) The *second-order analysis* shall consider both P-δ and P-Δ *effects*. It is permitted to perform the analysis using any general second-order analysis method, or by the amplified *first-order analysis* method of Section C2, provided that the B_1 and B_2 factors are based on the reduced *stiffness*es defined in Equations A-7-2 and A-7-3. Analyses shall be conducted according to the design and loading requirements specified in either Section B3.3 (LRFD) or Section B3.4 (ASD). For ASD, the second-order analysis shall be carried out under 1.6 times the ASD *load combinations* and the results shall be divided by 1.6 to obtain the *required strengths*.

Methods of analysis that neglect the effects of P-δ on the lateral displacement of the structure are permitted where the axial *loads* in all members whose flexural stiffnesses are considered to contribute to the lateral *stability* of the structure satisfy the following limit:

$$\alpha P_r < 0.15 P_{eL} \tag{A-7-1}$$

where

P_r = required axial compressive strength under LRFD or ASD *load combinations*, kips (N)
$P_{eL} = \pi^2 EI/L^2$, evaluated in the plane of bending

and

$$\alpha = 1.0 \text{ (LRFD)} \qquad \alpha = 1.6 \text{ (ASD)}$$

(2) A *notional load*, $N_i = 0.002Y_i$, applied independently in two orthogonal directions, shall be applied as a *lateral load* in all load combinations. This load shall be in addition to other lateral loads, if any,

where

N_i = notional lateral load applied at level i, kips (N)
Y_i = *gravity load* from the *LRFD load combination* or 1.6 times the ASD load combination applied at level i, kips (N)

The notional load coefficient of 0.002 is based on an assumed initial story out-of-plumbness ratio of 1/500. Where a smaller assumed out-of-plumbness is justified, the notional load coefficient may be adjusted proportionally.

For frames where the ratio of second-order drift to first-order drift is equal to or less than 1.5, it is permissible to apply the notional load, N_i, as a minimum lateral load for the gravity-only load combinations and not in combination with other lateral loads.

For all cases, it is permissible to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying a notional load or a minimum lateral load as defined above.

User Note: The unreduced stiffnesses (EI and AE) are used in the above calculations. The ratio of second-order drift to first-order drift can be represented by B_2, as calculated using Equation C2-3. Alternatively, the ratio can be calculated by comparing the results of a second-order analysis to the results of a first-order analysis, where the analyses are conducted either under LRFD load combinations directly or under ASD load combinations with a 1.6 factor applied to the ASD gravity loads.

(3) A reduced flexural *stiffness*, EI^*,

$$EI^* = 0.8\tau_b EI \tag{A-7-2}$$

shall be used for all members whose flexural stiffness is considered to contribute to the lateral *stability* of the structure,

where

I = moment of inertia about the axis of bending, in.^{4} (mm^4)
τ_b = 1.0 for $\alpha P_r/P_y \leq 0.5$
= $4[\alpha P_r/P_y\ (1-\alpha P_r/P_y)]$ for $\alpha P_r/P_y > 0.5$
P_r = required axial compressive strength under *LRFD* or *ASD load combinations*, kips (N)
P_y = AF_y, member *yield strength*, kips (N)

and

$$\alpha = 1.0 \text{ (LRFD)} \qquad \alpha = 1.6 \text{ (ASD)}$$

In lieu of using $\tau_b < 1.0$ where $\alpha P_r/P_y > 0.5$, $\tau_b = 1.0$ may be used for all members, provided that an additive *notional load* of $0.001Y_i$ is added to the notional load required in (2).

(4) A reduced axial *stiffness*, EA^*,

$$EA^* = 0.8EA \tag{A-7-3}$$

shall be used for members whose axial stiffness is considered to contribute to the lateral *stability* of the structure, where A is the cross-sectional member area.

COMMENTARY
on the Specification for Structural Steel Buildings

March 9, 2005

(The Commentary is not a part of ANSI/AISC 360-05, *Specification for Structural Steel Buildings*, but is included for informational purposes only.)

INTRODUCTION

The Specification is intended to be complete for normal design usage.

The Commentary furnishes background information and references for the benefit of the design professional seeking further understanding of the basis, derivations and limits of the specification.

The Specification and Commentary are intended for use by design professionals with demonstrated engineering competence.

Commentary Glossary

The Commentary uses the following terms in addition to the terms defined in the Glossary of the Specification. Only the terms listed below are *italicized* where they first appear in the Commentary text.

Alignment chart. Nomograph for determining the effective length factor K for some types of columns.

Biaxial bending. Simultaneous bending of a member about two perpendicular axes.

Brittle fracture. Abrupt cleavage with little or no prior ductile deformation.

Column curve. Curve expressing the relationship between axial column strength and slenderness ratio.

Critical load. Load at which a perfectly straight member under compression may either assume a deflected position or may remain undeflected, or a beam under flexure may either deflect and twist out of plane or remain in its in-plane deflected position, as determined by a theoretical stability analysis.

Cyclic load. Repeatedly applied external load that may subject the structure to fatigue.

Drift damage index. Parameter used to measure the potential damage caused by interstory drift.

Effective moment of inertia. Moment of inertia of the cross section of a member that remains elastic when partial plastification of the cross section takes place, usually under the combination of residual stress and applied stress. Also, the moment of inertia based on effective widths of elements that buckle locally. Also, the moment of inertia used in the design of partially composite members.

Effective stiffness. Stiffness of a member computed using the effective moment of inertia of its cross section.

Fatigue threshold. Stress range at which fatigue cracking will not initiate regardless of the number of cycles of loading.

First order plastic analysis. *Structural analysis* based on the assumption of rigid-plastic behavior—in other words, that equilibrium is satisfied throughout the structure and the stress is at or below the yield stress—and in which equilibrium conditions are formulated on the undeformed structure.

Flexible connection. Connection permitting a portion, but not all, of the simple beam rotation of a member end.

Flexural-torsional buckling. Buckling mode in which a compression member bends and twists simultaneously without change in cross-sectional shape.

Inelastic action. Material deformation that does not disappear on removal of the force that produced it.

Inelastic strength. Strength of a structure or component after material has achieved the *yield stress* at sufficient locations that a strength *limit state* is reached.

Interstory drift. Lateral deflection of a floor relative to the lateral deflection of the floor immediately below, divided by the distance between floors, $(\delta_n - \delta_{n-1})/h)$.

Permanent load. *Load* in which variations over time are rare or of small magnitude. All other *loads* are *variable loads*.

Primary member. For ponding analysis, beam or girder that supports the concentrated reactions from the secondary members framing into it.

Residual stress. Stress that remains in an unloaded member after it has been formed into a finished product. (Examples of such stresses include, but are not limited to, those induced by cold bending, cooling after rolling or welding).

Rigid frame. Structure in which connections maintain the angular relationship between beam and column members under load.

Secondary member. For ponding analysis, beam or joist that directly supports the distributed ponding loads on the roof of the structure.

Sidesway. Lateral movement of a structure under the action of lateral loads, unsymmetrical vertical loads or unsymmetrical properties of the structure.

Sidesway buckling. Buckling mode of a multistory frame precipitated by the relative lateral displacements of joints, leading to failure by sidesway of the frame.

Squash load. Column area multiplied by the yield stress.

St. Venant torsion. Portion of the torsion in a member that induces only shear stresses in the member.

Strain hardening. Phenomenon wherein ductile steel, after undergoing considerable deformation at or just above yield point, exhibits the capacity to resist substantially higher loading than that which caused initial yielding.

Subassemblage. Truncated portion of a structural frame.

Tangent modulus. At any given stress level, the slope of the stress-strain curve of a material in the inelastic range as determined by the compression test of a small specimen under controlled conditions.

Total building drift. Lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ/H.

Undercut. Notch resulting from the melting and removal of base metal at the edge of a weld.

Variable load. Load with substantial variation over time.

Warping torsion. Portion of the total resistance to torsion that is provided by resistance to warping of the cross section.

Yield plateau. Portion of the stress-strain curve for uniaxial tension or compression in which the stress remains essentially constant during a period of substantially increased strain.

CHAPTER A

GENERAL PROVISIONS

A1. SCOPE

The scope of this Specification is broader than that of the two AISC Specifications that it replaces: the 1999 *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 2000b) and the 1989 *ASD Specification* (AISC, 1989). This Specification combines these two previous Specifications and incorporates the provisions of the *Load and Resistance Factor Design Specification for Steel Hollow Structural Sections* (AISC, 2000), the *Specification for Allowable Stress Design of Single-Angle Members* (AISC, 1989) and the *Load and Resistance Factor Design Specification for Single-Angle Members* (AISC, 2000a). The basic purpose of the provisions in this Specification is the determination of the available and nominal strength of the members, connections and other components of steel building structures. The nominal strength is usually defined in terms of resistance to a load effect, such as axial force, bending moment, shear or torque, but in some instances it is expressed in terms of a stress.

This Specification provides two methods of design:

(1) **Load and Resistance Factor Design (LRFD):** The nominal strength is multiplied by a resistance factor ϕ, and the resulting design strength is then required to equal or exceed the required strength determined by structural analysis for the appropriate LRFD load combination specified by the applicable building code.

(2) **Allowable Strength Design (ASD):** The nominal strength is divided by a safety factor Ω, and the resulting allowable strength is then required to equal or exceed the required strength determined by structural analysis for the appropriate ASD load combination specified by the applicable building code.

This Specification gives provisions for determining the values of the nominal strengths according to the applicable limit states and lists the corresponding values of the resistance factor ϕ and the safety factor Ω. The ASD safety factors are calibrated to give the same structural reliability and the same component size as the LRFD method at a live-to-dead load ratio of 3.

This Specification is applicable to both buildings and other structures. Many structures found in petrochemical plants, power plants, and other industrial applications are designed, fabricated and erected in a manner similar to buildings. It is not intended that this Specification address steel structures with vertical and lateral load-resisting systems that are not similar to buildings, such as those constructed of shells or catenary cables.

For the purposes of this Specification, HSS are defined as hollow structural sections with constant wall thickness and a round, square or rectangular cross section that is constant along the length of the member. HSS are manufactured by forming skelp (strip or plate) to the desired shape and joining the edges with a continuously welded seam. Published information is available describing the details of the various methods used to manufacture HSS (Graham, 1965; STI, 1996).

The *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) defines the practices that are the commonly accepted standards of custom and usage for structural steel fabrication and erection. As such, the *Code of Standard Practice* is primarily intended to serve as a contractual document to be incorporated into the contract between the buyer and seller of fabricated structural steel. Some parts of the *Code of Standard Practice*, however, form the basis for some of the provisions in this Specification. Therefore, the *Code of Standard Practice* is referenced in selected locations in this Specification to maintain the ties between these documents, where appropriate.

A2. REFERENCED SPECIFICATIONS, CODES AND STANDARDS

Section A2 provides references to documents cited in this Specification. Note that not all grades of a particular material specification are necessarily approved for use according to this Specification. For a list of approved materials and grades, see Section A3.

A3. MATERIAL

1. Structural Steel Materials

1a. ASTM Designations

There are hundreds of steel materials and products. This Specification lists those products/materials that are commonly useful to structural engineers and those that have a history of satisfactory performance. Other materials may be suitable for specific applications, but the evaluation of those materials is the responsibility of the engineer specifying them. In addition to typical strength properties, considerations for materials may include but are not limited to strength properties in transverse directions, ductility, formability, soundness, weldability including sensitivity to thermal cycles, notch toughness and other forms of crack sensitivity, coatings and corrosivity. Consideration for product form may include material considerations in addition to effects of production, tolerances, testing, reporting and surface profiles.

Hot-Rolled Structural Shapes. The grades of steel approved for use under this Specification, covered by ASTM specifications, extend to a yield stress of 100 ksi (690 MPa). Some of the ASTM specifications specify a minimum yield point, while others specify a minimum yield strength. The term "yield stress" is used in this Specification as a generic term to denote either the yield point or the yield strength.

It is important to be aware of limitations of availability that may exist for some combinations of strength and size. Not all structural section sizes are included in the various material specifications. For example, the 60 ksi (415 MPa) yield stress steel in the A572/A572M specification includes plate only up to $1^1/_4$ in. (32 mm) in thickness. Another limitation on availability is that even when a product is included in this Specification, it may be infrequently produced by the mills. Specifying these products may result in procurement delays or require ordering large quantities directly from the producing mills. Consequently, it is prudent to check availability before completing the details of a design. The AISC web site provides this information (www.aisc.org) and AISC's *Modern Steel Construction* publishes tables on availability twice per year.

Properties in the direction of rolling are of principal interest in the design of steel structures. Hence, yield stress as determined by the standard tensile test is the principal mechanical property recognized in the selection of the steels approved for use under this Specification. It must be recognized that other mechanical and physical properties of rolled steel, such as anisotropy, ductility, notch toughness, formability, corrosion resistance, etc., may also be important to the satisfactory performance of a structure.

It is not possible to incorporate in the Commentary adequate information to impart full understanding of all factors that might merit consideration in the selection and specification of materials for unique or especially demanding applications. In such a situation the user of the Specification is advised to make use of reference material contained in the literature on the specific properties of concern and to specify supplementary material production or quality requirements as provided for in ASTM material specifications. One such case is the design of highly restrained welded connections (AISC, 1973). Rolled steel is anisotropic, especially insofar as ductility is concerned; therefore, weld contraction strains in the region of highly restrained welded connections may exceed the strength of the material if special attention is not given to material selection, details, workmanship and inspection.

Another special situation is that of fracture control design for certain types of service conditions (AASHTO, 1998). For especially demanding service conditions such as structures exposed to low temperatures, particularly those with impact loading, the specification of steels with superior notch toughness may be warranted. However, for most buildings, the steel is relatively warm, strain rates are essentially static, and the stress intensity and number of cycles of full design stress are low. Accordingly, the probability of fracture in most building structures is low. Good workmanship and good design details incorporating joint geometry that avoids severe stress concentrations are generally the most effective means of providing fracture-resistant construction.

Hollow Structural Sections (HSS). Specified minimum tensile properties are summarized in Table C-A3.1 for various HSS and pipe material specifications and grades. ASTM A53 Grade B is included as an approved pipe material

TABLE C-A3.1
Minimum Tensile Properties of HSS and Pipe Steels

Specification	Grade	F_y, ksi (MPa)	F_u, ksi (MPa)
ASTM A53	**B**	35 (240)	60 (415)
ASTM A500 (round)	**A**	33 (228)	45 (311)
	B	42 (290)	58 (400)
	C	46 (317)	62 (428)
ASTM A500 (rectangular)	**A**	39 (269)	45 (311)
	B	46 (317)	58 (400)
	C	50 (345)	62 (428)
ASTM A501	**–**	36 (248)	58 (400)
ASTM A618 (round)	**I and II**	50 (345)	70 (483)
	III	50 (345)	65 (450)
ASTM A847	**–**	50 (345)	70 (483)
CAN/CSA-G40.20/G40.21	**350W**	51 (350)	65 (450)

specification because it is the most readily available round product in the United States. Other North American HSS products that have properties and characteristics that are similar to the approved ASTM products are produced in Canada under the *General Requirements for Rolled or Welded Structural Quality Steel* (CSA, 2003). In addition, pipe is produced to other specifications that meet the strength, ductility and weldability requirements of the materials in Section A3, but may have additional requirements for notch toughness or pressure testing.

Pipe can be readily obtained in ASTM A53 material and round HSS in ASTM A500 Grade B is also common. For rectangular HSS, ASTM A500 Grade B is the most commonly available material and a special order would be required for any other material. Depending upon size, either welded or seamless round HSS can be obtained. In North America, however, all ASTM A500 rectangular HSS for structural purposes are welded. Rectangular HSS differ from box sections in that they have uniform thickness except for some thickening in the rounded corners.

ASTM A500 Grade A material does not meet the ductility “limit of applicability” for direct connections in Section K2.3a(12). This limit requires that $F_y/F_u \leq 0.8$. In determining that other materials meet the ductility limit, it is important to note that ASTM A500 permits the yield strength to be determined by either the 0.2 percent offset method or at 0.5 percent elongation under load (EUL). Since ASTM A500 materials are cold-formed and have rounded stress-strain curves with no *yield plateau*, the latter method indicates yield strengths greater than the 0.2 percent offset. The ductility limit is intended to apply to yield strengths determined by the 0.2 percent offset. However, mill reports may indicate the EUL yield, raising concerns that the material does not have adequate ductility. Supplemental tension tests may be required to determine the 0.2 percent offset yield strength.

Even though ASTM A501 includes rectangular HSS, hot-formed rectangular HSS are not currently produced in the United States. The *General Requirements for Rolled or Welded Structural Quality Steel* (CSA, 2003) includes Class C (cold-formed) and Class H (cold-formed and stress relieved) HSS. Class H HSS have relatively low levels of *residual stress*, which enhances their performance in compression and may provide better ductility in the corners of rectangular HSS.

1c. Rolled Heavy Shapes

The web-to-flange intersection and the web center of heavy hot-rolled shapes, as well as the interior portions of heavy plates, may contain a more coarse grain structure and/or lower notch toughness material than other areas of these products. This is probably caused by ingot segregation, the somewhat lesser deformation during hot rolling, higher finishing temperature, and the slower cooling rate after rolling for these heavy sections. This characteristic is not detrimental to suitability for compression members or for nonwelded members. However, when heavy cross sections are joined by splices or connections using complete-joint-penetration welds that extend through the coarser and/or lower notch-tough interior portions, tensile strains induced by weld shrinkage may result in cracking. An example is a complete-joint-penetration groove welded connection of a heavy cross section beam to any column section. When members of lesser thickness are joined by complete-joint-penetration groove welds, which induce smaller weld shrinkage strains, to the finer grained and/or more notch-tough surface material of ASTM A6/A6M shapes and heavy built-up cross sections, the potential for cracking is significantly lower. An example is a complete-joint-penetration groove welded connection of a nonheavy cross-section beam to a heavy cross-section column.

For critical applications such as primary tension members, material should be specified to provide adequate notch toughness at service temperatures. Because of differences in the strain rate between the Charpy V-Notch (CVN) impact test and the strain rate experienced in actual structures, the CVN test is conducted at a temperature higher than the anticipated service temperature for the structure. The location of the CVN test specimens (“alternate core location”) is specified in ASTM A6/A6M, Supplemental Requirement S30.

The notch toughness requirements of Section A3.1c are intended only to provide material of reasonable notch toughness for ordinary service applications. For unusual applications and/or low temperature service, more restrictive requirements and/or notch toughness requirements for other section sizes and thicknesses may be appropriate. To minimize the potential for fracture, the notch toughness requirements of Section A3.1c must be used in conjunction with good design and fabrication procedures. Specific requirements are given in Sections J1.5, J1.6, J2.6 and J2.7.

For rotary-straightened W-shapes, an area of reduced notch toughness has been documented in a limited region of the web immediately adjacent to the flange. This region may exist in W-shapes of all weights, not just heavy shapes. Considerations in design and detailing that recognize this situation are presented in Chapter J.

2. Steel Castings and Forgings

There are a number of ASTM specifications for steel castings. The SFSA *Steel Castings Handbook* (SFSA, 1995) recommends ASTM A216 as a product useful for steel structures. In addition to the requirements of this Specification, SFSA recommends that various other requirements be considered for cast steel products. It may be appropriate to inspect the first piece cast using magnetic particle inspection in accordance with ASTM E125, degree 1a, b, or c. Radiographic inspection level III may be desirable for critical sections of the first piece cast. Ultrasonic testing (UT) in compliance with ASTM E609 may be appropriate for first cast piece over 6 in. thick. Design approval, sample approval, periodic nondestructive testing of the mechanical properties, chemical testing, and selection of the correct welding specification should be among the issues defined in the selection and procurement of cast steel products. Refer to SFSA (1995) for design information about cast steel products.

3. Bolts, Washers and Nuts

The ASTM standard specification for A307 bolts covers two grades of fasteners. Either grade may be used under this Specification; however, it should be noted that Grade B is intended for pipe flange bolting and Grade A is the grade long in use for structural applications.

4. Anchor Rods and Threaded Rods

ASTM F1554 is the primary specification for anchor rods. Since there is a limit on the maximum available length of ASTM A325/A325M and ASTM A490/A490M bolts, the attempt to use these bolts for anchor rods with design lengths longer than the maximum available lengths has presented problems in the past. The inclusion of ASTM A449 and A354 materials in this Specification allows the use of higher strength material for bolts longer than ASTM A325/A325M and ASTM A490/A490M bolts.

The engineer of record should specify the required strength for threaded rods used as load-carrying members.

5. Filler Metal and Flux for Welding

The AWS Filler Metal Specifications listed in Section A3.5 are general specifications that include filler metal classifications suitable for building construction, as well as classifications that may not be suitable for building construction. The AWS D1.1, *Structural Welding Code Steel* (AWS, 2004) lists in Table 3.1 various electrodes that may be used for prequalified welding procedure specifications, for the various steels that are to be joined. This list specifically does not include various classifications of filler metals that are not suitable for structural steel applications. Filler metals listed under the various AWS A5 specifications may or may not have specified notch toughness properties, depending on the specific electrode classification. Section J2.6 identifies certain welded joints where notch toughness of filler metal is needed in building construction. There may be other situations where the engineer of record may elect to specify the use of filler metals with specified notch toughness properties, such as for structures subject to high loading rate, cyclic loading or seismic loading. Since AWS D1.1 does not automatically require that the filler metal used have specified notch toughness properties, it is important that filler metals used for such applications be of an AWS classification where such properties are required. This information can be found in the AWS Filler Metal Specifications and is often contained on the filler metal manufacturer's certificate of conformance or product specification sheets.

When specifying filler metal and/or flux by AWS designation, the applicable standard specifications should be carefully reviewed to assure a complete understanding of the designation reference. This is necessary because the AWS designation systems are not consistent. For example, in the case of electrodes for shielded metal arc welding (AWS A5.1), the first two or three digits indicate the nominal tensile strength classification, in ksi, of the filler metal and the final two digits indicate the type of coating. For metric designations, the first two digits times 10 indicate the nominal tensile strength classification in MPa. In the case of mild steel electrodes for submerged arc welding (AWS A5.17), the first one or two digits times 10 indicate the nominal tensile strength classification for both U.S. customary and metric units, while the final digit or digits times 10 indicate the testing temperature in degrees F, for filler metal impact tests. In the case of low-alloy steel covered arc welding electrodes (AWS A5.5), certain portions of the designation indicate a requirement for stress relief, while others indicate no stress relief requirement.

Engineers do not, in general, specify the exact filler metal to be employed on a particular structure. Rather, the decision as to which welding process and which filler metal are to be utilized is usually left with the fabricator or erector. Codes restrict the usage of certain filler materials, or impose qualification testing to prove the suitability of the specific electrode, so as to make certain that the proper filler metals are used.

A4. STRUCTURAL DESIGN DRAWINGS AND SPECIFICATIONS

The abbreviated list of requirements in this Specification is intended to be compatible with and a summary of the more extensive requirements in Section 3 of the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005). The user should refer to Section 3 of the *Code of Standard Practice for Steel Buildings and Bridges* for further information.

CHAPTER B

DESIGN REQUIREMENTS

B1. GENERAL PROVISIONS

Previous editions of the Specification contained a section entitled “Types of Construction,” for example, Section A2 in the 1999 *LRFD Specification* (AISC, 2000b). In this Specification there is no such section and the requirements related to “types of construction” have been divided between Section B1, Section B3.6, and Section J1.

Historically, “Types of Construction” was the section that established what type of structures the Specification covers. The preface to the 1999 *LRFD Specification* (AISC, 2000b) suggests that the purpose of the Specification is “to provide design criteria for routine use and not to provide specific criteria for infrequently encountered problems.” The preface to the 1978 *Specification* (AISC, 1978) contains similar language. While “routine use” may be difficult to describe, the contents of “Types of Construction” have been clearly directed at ordinary building frames with beams, columns and their connections.

The 1969 *Specification* (AISC, 1969) classified “types of construction” as Type 1, 2 or 3. The primary distinction among these three types of construction was the nature of the connections of the beams to the columns. Type 1 construction comprised “*rigid frames*,” now called moment-resisting frames that had connections capable of transmitting moment. Type 2 construction comprised “simple frames” with no moment transfer between beams and columns. Type 3 construction comprised “semi-rigid frames.” Type 3 construction used partially restrained connections and was allowed if a predictable and reliable amount of connection flexibility and moment transfer was demonstrable.

The 1986 *LRFD Specification* (AISC, 1986) changed the designation from Type 1, 2 or 3 to the designations FR (Fully Restrained) and PR (Partially Restrained). In these designations the term “restraint” refers to the degree of moment transfer and the associated deformation in the connections. The 1986 *LRFD Specification* also used the term “simple framing” to refer to structures with “simple connections,” that is, connections with negligible moment transfer. In essence, FR was equivalent to Type 1, “simple framing” was equivalent to Type 2, and PR was equivalent to Type 3 construction.

Type 2 construction of earlier specifications and “simple framing” of the 1986 *LRFD Specification* had additional provisions that allowed the wind loads to be carried by moment resistance of selected joints of the frame provided that:

(1) The connections and connected members have capacity to resist the wind moments;
(2) The girders are adequate to carry the full gravity load as "simple beams"; and
(3) The connections have adequate inelastic rotation capacity to avoid overstress of the fasteners or welds under combined gravity and wind loading.

The justification of considering the so-called "wind connections" as both simple (for gravity loads) and moment resisting (for wind loads) was provided in Sourochnikoff (1950) and Disque (1964). The basic argument asserts that the connections actually have some moment resistance but that the strength is low enough that under wind loads the connections would sustain inelastic deformations. Under repeated wind loads, then, the connection response would "shake down" to a condition wherein the moments in the connections under gravity loads would be very small but the elastic resistance of the connections to wind moments would remain the same as the initial resistance. These additional provisions for Type 2 construction have been used successfully for many years. More recent recommendations for this type of system are provided in Geschwindner and Disque (2005).

Section B1 widens the purview of this Specification to a broader class of construction types. It recognizes that a structural system is a combination of members connected in such a way that the structure can respond in different ways to meet different design objectives under different loads. Even within the purview of ordinary buildings, there can be an enormous variety in the design details.

This Specification is still meant to be primarily applicable to the common types of building frames with gravity loads carried by beams and girders and lateral loads carried by moment frames, braced frames or shear walls. However, there are many unusual buildings for which this Specification is also applicable. Rather than to attempt to establish the purview of the Specification with an exhaustive classification of construction types, Section B1 requires that the design of members and their connections be consistent with the intended use of the structure and the assumptions made in the analysis of the structure.

B2. LOADS AND LOAD COMBINATIONS

The loads and load combinations for this Specification are given in the applicable building code. In the absence of a specific local, regional or national building code, the load combinations and the nominal loads (for example, D, L, L_r, S, R, W and E) are the loads specified in Sections 3 through 9 of SEI/ASCE 7, *Minimum Design Loads for Buildings and Other Structures* (ASCE, 2002). The latest 2002 edition of SEI/ASCE 7 has adopted, in most aspects, the seismic design provisions from the NEHRP *Recommended Provisions* (NEHRP, 1997), as have the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2002). The reader is referred to the commentaries of these documents for an expanded discussion on loads, load factors and seismic design.

This Specification permits design for strength by either LRFD or ASD.

LRFD Load Combinations. If LRFD is selected, the load combination requirements are defined in Section 2.3 of SEI/ASCE 7, while if ASD is selected, the load combination requirements are defined in Section 2.4 of that standard. In either case, it is assumed that the nominal loads—D, L, L_r, S, R, W and E—are as specified in Sections 3 through 9 of SEI/ASCE 7, or their equivalent, as stipulated by the authority having jurisdiction. The engineer should understand that the bases for the load combinations in Sections 2.3 and 2.4 of SEI/ASCE 7 are different.

The load combinations in Section 2.3 of SEI/ASCE 7 are based on modern probabilistic load modeling and a comprehensive survey of reliabilities inherent in traditional design practice (Galambos, Ellingwood, MacGregor, and Cornell, 1982; Ellingwood, MacGregor, Galambos, and Cornell, 1982). These load combinations utilize a "principal action-companion action format," which is based on the notion that the maximum combined load effect occurs when one of the time-varying loads takes on its maximum lifetime value (principal action) while the other *variable loads* are at "arbitrary point-in-time" values (companion actions), the latter being loads that would be measured in a load survey at any arbitrary time. The dead load, which is considered to be permanent, is the same for all combinations in which the load effects are additive. Research has shown that this approach to load combination analysis is consistent with the manner in which loads actually combine on structural elements and systems in situations in which strength limit states may be approached. The load factors reflect uncertainty in individual load magnitudes and in the analysis that transforms load to load effect. The nominal loads in Sections 3 through 9 of SEI/ASCE 7 are substantially in excess of the arbitrary point-in-time values. The nominal live, wind and snow loads historically have been associated with mean return periods of approximately 50 years, while the nominal earthquake effect in NEHRP (1997) is associated with a mean return period of approximately 2,500 years. To avoid having to specify both a maximum and an arbitrary point-in-time value for each load type, some of the specified load factors are less than unity in SEI/ASCE 7 combinations (2) through (5).

Load combinations (6) and (7) of SEI/ASCE 7, Section 2.3, apply specifically to cases in which the structural actions due to lateral forces and gravity loads counteract one another. In that case, where the dead load stabilizes the structure, the load factor on dead load is 0.9.

ASD Load Combinations. The load combinations in Section 2.4 of SEI/ASCE 7 for ASD are similar to those that have been used in allowable stress design for the past four decades. In ASD, safety is provided by the safety factor, Ω, and the nominal loads in the basic combinations (1) through (3) are not factored. The reduction in the combined time-varying load effect in combinations (4) and (6) is achieved by the load combination factor 0.75. This load combination factor dates back to the 1972 edition of ANSI Standard A58.1, the predecessor of SEI/ASCE 7. It should be noted that in SEI/ASCE 7, the 0.75 factor applies *only* to combinations of *variable loads*; it is irrational to reduce the dead load because it is *always* present and does not fluctuate in time. The load factor $0.6D$ in load combinations (7) and (8) in Section 2.4 of SEI/ASCE 7 addresses the situation in which the effects of lateral or uplift forces counteract the effect of gravity loads. This eliminates a deficiency in the traditional treatment of counteracting loads in allowable stress design and emphasizes the importance of checking stability. The earthquake load effect is multiplied by 0.7 in combinations (5) and (8) to align allowable strength design for earthquake effects with the definition of E in Section 9 of SEI/ASCE 7 which is based on strength principles.

The load combinations in Sections 2.3 and 2.4 of SEI/ASCE 7 apply only to design for strength limit states. Neither of these account for gross error or negligence.

Serviceability Load Combinations. Serviceability limit states and associated load factors are covered in Appendix B of SEI/ASCE 7. That Appendix contains a number of suggested load combinations for checking serviceability. While the nominal loads appearing in those equations are defined in Sections 3 through 7 of SEI/ASCE 7, the performance objectives for serviceability checking are different from those for checking strength, and thus the combinations and load factors are different.

B3. DESIGN BASIS

Load and Resistance Factor Design (LRFD) and Allowable Strength Design (ASD) are distinct methods. They are equally acceptable by this Specification, but their provisions are not identical and not interchangeable. Indiscriminate use of combinations of the two methods could result in design error. For these reasons they are specified as alternatives. There are, however, circumstances in which the two methods could be used in the design, modification or renovation of a structural system without conflicting, such as providing modifications to a structural floor system of an older building after assessing the as-built conditions.

1. Required Strength

This Specification permits the use of elastic, inelastic or plastic structural analysis. Generally, design is performed by elastic analysis. Provisions for inelastic and plastic analysis are given in Appendix 1. The required strength is determined by the appropriate methods of structural analysis.

In some circumstances, as in the proportioning of stability bracing members that carry no calculated forces (see, for example, Appendix 6), the required strength is explicitly stated in this Specification.

2. Limit States

A limit state is a condition in which a structural system or component becomes unfit for its intended purpose, when it is exceeded. Limit states may be dictated by functional requirements, such as maximum deflections or drift; they may be related to structural behavior, such as the formation of a plastic hinge or mechanism;

or they may represent the collapse of the whole or part of the structure, such as by instability or fracture. The design provisions provided make certain that the probability of reaching a limit state is acceptably small by stipulating the combination of load factors, resistance or safety factors, nominal loads and nominal strengths consistent with the design assumptions.

Two kinds of limit states apply to structures: (1) strength limit states define safety against local or overall failure conditions during the intended life of the structure; and (2) serviceability limit states define functional requirements. This Specification, like other structural design codes, primarily focuses on strength limit states because of overriding considerations of public safety. This does not mean that limit states of serviceability are not important to the designer, who must provide for functional performance and economy of design. However, serviceability considerations permit more exercise of judgment on the part of the designer.

Strength limit states vary from element to element, and several limit states may apply to a given element. The following strength limit states are the most common: yielding, formation of a plastic hinge, member or overall frame instability, lateral-torsional buckling, local buckling, rupture and fatigue. The most common serviceability limit states include unacceptable elastic deflections and drift, unacceptable vibrations, and permanent deformations.

3. Design for Strength Using Load and Resistance Factor Design (LRFD)

Design for strength by LRFD is performed in accordance with Equation B3-1. The left side of Equation B3-1, R_u, represents the required strength computed by structural analysis based on loads stipulated in SEI/ASCE 7 (ASCE, 2002), Section 2.3 (or their equivalent), while the right side, ϕR_n, represents the limiting structural resistance, or *design strength*, provided by the member.

The resistance factor ϕ in this Specification is equal to or less than 1.0. When compared to the nominal strength, R_n, computed according to the methods given in Chapters D through K, a ϕ-value of less than 1.0 accounts for inaccuracies of the theory and variations in mechanical properties and dimensions of members and frames. For limit states where $\phi = 1.0$, the nominal strength is judged to be sufficiently conservative when compared to the actual strength that no further reduction is needed.

The LRFD provisions are based on: (1) probabilistic models of loads and resistance; (2) a calibration of the LRFD provisions to the 1978 edition of the ASD Specification for selected members; and (3) the evaluation of the resulting provisions by judgment and past experience aided by comparative design office studies of representative structures.

In the probabilistic basis for LRFD (Ravindra and Galambos, 1978; Ellingwood and others, 1982), the load effects Q and the resistances R are modeled as statistically independent random variables. In Figure C-B3.1, relative frequency distributions for Q and R are portrayed as separate curves on a common plot for a hypothetical case. As long as the resistance R is greater than (to the right of) the effects of the loads Q, a margin of safety for the particular limit state exists. However, because Q and R are random variables, there is a small probability that R may be less than Q, in other words, $R < Q$. The probability of this limit state is related to the degree of overlap of the frequency distributions in Figure C-B3.1, which depends on their relative positioning (R_m *versus* Q_m) and their dispersions.

The probability that R is less than Q depends on the distribution shapes of each of the many variables (material, loads, etc.) that determine resistance and total load effect. Often, only the means and the standard deviations or coefficients of variation of the many variables involved in the makeup of R and Q can be estimated. However, this information is sufficient to build an approximate design provision that is independent of the knowledge of these distributions, by stipulating the following design condition:

$$\beta\sqrt{V_R^2 + V_Q^2} \le \ln\left(R_m / Q_m\right) \tag{C-B3-1}$$

In this equation, R_m and Q_m are the mean values and V_R and V_Q are the coefficients of variation, respectively, of the resistance R and the load effect Q. For structural elements and the usual loading, R_m, Q_m, and the coefficients of variation, V_R and V_Q, can be estimated, so a calculation of

$$\beta = \frac{\ln\left(R_m / Q_m\right)}{\sqrt{V_R^2 + V_Q^2}} \tag{C-B3-2}$$

will give a comparative value of the measure of reliability of a structure or component. The parameter, β, is denoted the "safety" or "reliability" index.

Extensions to the determination of β in Equation C-B3-2 to accommodate additional probabilistic information and more complex design situations are described in Ellingwood and others (1982) and have been used in the development of the recommended load combinations in SEI/ASCE 7.

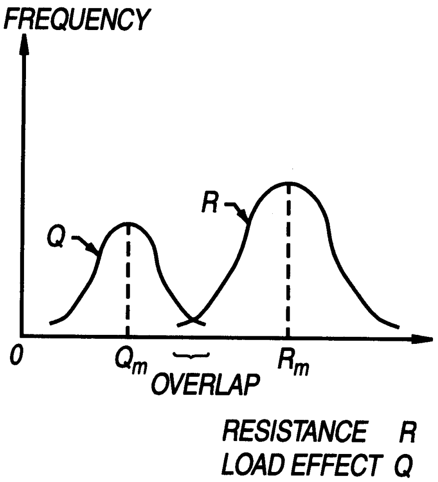


Fig. C-B3.1. Frequency distribution of load effect Q and resistance R.

The original studies for the statistical properties (mean values and coefficients of variation) used to develop the LRFD provisions for the basic material properties and for steel beams, columns, composite beams, plate girders, beam-columns and connection elements are presented in a series of eight articles in the September 1978 issue of the *Journal of the Structural Division*, ASCE (Vol. 104, ST9). The corresponding load statistics are given in Galambos and others (1982). Based on these statistics, the values of β inherent in the 1978 *Specification* (AISC, 1978) were evaluated under different load combinations (live/dead, wind/dead, etc.) and for various tributary areas for typical members (beams, columns, beam-columns, structural components, etc.). As might be expected, there was a considerable variation in the range of β-values. For example, compact rolled beams (flexure) and tension members (yielding) had β-values that decreased from about 3.1 at $L/D = 0.50$ to 2.4 at $L/D = 4$. This decrease is a result of ASD applying the same factor to dead load, which is relatively predictable, and live load, which is more variable. For bolted or welded connections, β was on the order of 4 to 5.

The variation of β that was inherent to ASD is reduced substantially in LRFD by specifying several target β-values and selecting load and resistance factors to meet these targets. The Committee on Specifications set the point at which LRFD is calibrated to ASD at $L/D = 3.0$ for braced compact beams in flexure and tension members at yield. The resistance factor, ϕ, for these limit states is 0.90, and the implied β is approximately 2.6 for members and 4.0 for connections. The larger β-value for connections reflects the fact that connections are expected to be stronger than the members they connect. Limit states for other members are handled similarly.

The databases on steel strength used in previous editions of the *LRFD Specification* were based mainly on research conducted prior to 1970. An important recent study of the material properties of structural shapes (Bartlett, Dexter, Graeser, Jelinek, Schmidt, and Galambos, 2003) addressed changes in steel production methods and steel materials that have occurred over the past 15 years. It was concluded that the new steel material characteristics did not warrant changes in the ϕ-values.

4. Design for Strength Using Allowable Strength Design (ASD)

The ASD method is provided in this Specification as an equal alternative to LRFD for use by engineers who prefer to deal with ASD load combinations and allowable stresses in the traditional ASD format. The term "allowable strength" has been introduced to emphasize that the basic equations of structural mechanics that underlie the provisions are the same for LRFD and ASD. This represents a departure from the past when LRFD and ASD were governed by separate specifications.

Traditional ASD is based on the concept that the maximum stress in a component shall not exceed a certain allowable stress under normal service conditions. The load effects are determined on the basis of an elastic analysis of the structure, while the allowable stress is the limiting stress (at yielding, instability, fracture, etc.) divided by a safety factor. The magnitude of the safety factor and the resulting allowable stress depend on the particular governing limit state against which the design must produce a certain margin of safety. For any single element, there may be a number of different allowable stresses that must be checked.

The safety factor in traditional ASD provisions was a function of both the material and the component being considered. It may have been influenced by factors such as member length, member behavior, load source and anticipated quality of workmanship. The traditional safety factors were based solely on experience and have remained unchanged for over 50 years. Although ASD-designed structures have performed adequately over the years, the actual level of safety provided was never known. This was the prime drawback of the traditional ASD approach. An illustration of typical performance data is provided in Bjorhovde (1978), where theoretical and actual safety factors for columns are examined.

Design for strength by ASD is performed in accordance with Equation B3-2. The ASD method provided in the Specification recognizes that the controlling modes of failure are the same for structures designed by ASD and LRFD. Thus, the nominal strength that forms the foundation of LRFD is the same nominal strength that provides the foundation for ASD. When considering available strength, the only difference between the two methods is the resistance factor in LRFD, ϕ, and the safety factor in ASD, Ω.

In developing appropriate values of Ω for use in this Specification, the aim was to assure similar levels of safety and reliability for the two methods. A straight forward approach for relating the resistance factor and the safety factor was developed. As already mentioned, the original LRFD Specification was calibrated to the 1978 *ASD Specification* at a live load to dead load ratio of 3. Thus, by equating the designs for the two methods at a ratio of live-to-dead load of 3, the relationship between ϕ and Ω can be determined. Using the live plus dead load combinations, with $L = 3D$, yields

$$\text{For LRFD}: \quad \phi R_n = 1.2D + 1.6L = 1.2D + 1.6 \times 3D = 6D \qquad \text{(C-B3-3)}$$

$$R_n = \frac{6D}{\phi}$$

$$\text{For ASD}: \quad \frac{R_n}{\Omega} = D + L = D + 3D = 4D \qquad \text{(C-B3-4)}$$

$$R_n = \frac{4D}{\Omega}$$

Equating R_n from the LRFD and ASD formulations and solving for Ω yields

$$\Omega = \frac{6D}{\phi} \times \frac{1}{4D} = \frac{1.5}{\phi} \qquad \text{(C-B3-5)}$$

A similar approach was used to obtain the majority of values of Ω throughout the Specification.

5. Design for Stability

Section B3.5 provides the charging language for Chapter C on design for stability.

6. Design of Connections

Section B3.6 provides the charging language for Chapter J on the design of connections. Chapter J covers the proportioning of the individual elements of a connection (angles, welds, bolts, etc.) once the load effects on the connection are known. Section B3.6 establishes that the modeling assumptions associated with the structural analysis must be consistent with the conditions used in Chapter J to proportion the connecting elements.

In many situations, it is not necessary to include the connection elements as part of the analysis of the structural system. For example, simple and FR connections may often be idealized as pinned or fixed, respectively, for the purposes of structural analysis. Once the analysis has been completed, the deformations or forces computed at the joints may be used to proportion the connection elements. The classification of FR (fully restrained) and simple connections is meant to justify these idealizations for analysis with the provision that if, for example, one assumes a connection to be FR for the purposes of analysis, then the actual connection must meet the FR conditions. In other words, it must have adequate strength and stiffness, as described in the provisions and discussed below.

In certain cases, the deformation of the connection elements affects the way the structure resists load and hence the connections must be included in the analysis of the structural system. These connections are referred to as partially restrained (PR) moment connections. For structures with PR connections, the connection flexibility must be estimated and included in the structural analysis, as described in the following sections. Once the analysis is complete, the load effects and deformations computed for the connection can be used to check the adequacy of the connecting elements.

For simple and FR connections, the connection proportions are established after the final analysis of the structural design is completed, thereby greatly simplifying the design cycle. In contrast, the design of PR connections (like member selection) is inherently iterative because one must assume values of the connection proportions in order to establish the force-deformation characteristics of the connection needed to perform the structural analysis. The life-cycle performance characteristics (shakedown) must also be considered. The adequacy of the assumed proportions of the connection elements can be verified once the outcome of the structural analysis is known. If the connection elements are inadequate, then the values must be revised and the structural analysis repeated. The potential benefits of using PR connections for various types of framing systems are discussed extensively in the literature [for example, Lorenz, Kato, and Chen (1993); Leon (1994)].

Connection Classification. The basic assumption made in classifying connections is that the most important behavioral characteristics of the connection can be modeled by a moment-rotation (M-θ) curve. Figure C-B3.2 shows a typical M-θ

curve. Implicit in the moment-rotation curve is the definition of the connection as being a region of the column and beam along with the connecting elements. The connection response is defined this way because the rotation of the member in a physical test is generally measured over a gage length that incorporates the contributions of not only the connecting elements, but also the ends of the members being connected and the column panel zone.

Examples of connection classification schemes include those in Bjorhovde, Colson, and Brozzetti (1990) and Eurocode 3 (1992). These classifications account directly for the stiffness, strength and ductility of the connections.

Connection Stiffness. Because the nonlinear behavior of the connection manifests itself even at low moment-rotation levels, the initial stiffness of the connection K_i (shown in Figure C-B3.2) does not adequately characterize connection response at service levels. Furthermore, many connection types do not exhibit a reliable initial stiffness, or it exists only for a very small moment-rotation range. The secant stiffness K_S at service loads is taken as an index property of connection stiffness. Specifically, $K_S = M_S/\theta_S$ where M_S and θ_S are the moment and rotation, respectively, at service loads. In the discussion below, L and EI are the length and bending rigidity, respectively, of the beam.

If $K_S L/EI \geq 20$, then it is acceptable to consider the connection to be fully restrained (in other words, able to maintain the angles between members). If $K_S L/EI \leq 2$, then it is acceptable to consider the connection to be simple (in other words, rotates without developing moment). Connections with stiffnesses between these two limits are partially restrained and the stiffness, strength and ductility of the connection must be considered in the design (Leon, 1994). Examples of FR, PR and simple connection response curves are shown in Figure C-B3.3. The solid dot θ_S reflects the service load level and thereby defines the secant stiffness.

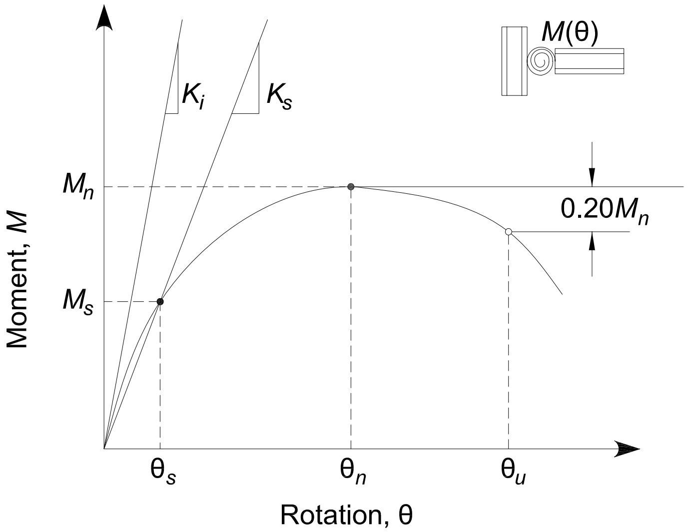


Fig. C-B3.2. Definition of stiffness, strength and ductility characteristics of the moment-rotation response of a partially restrained connection.

Connection Strength. The strength of a connection is the maximum moment that it is capable of carrying M_n, as shown in Figure C-B3.2. The strength of a connection can be determined on the basis of an ultimate limit-state model of the connection, or from a physical test. If the moment-rotation response does not exhibit a peak load then the strength can be taken as the moment at a rotation of 0.02 radian (Hsieh and Deierlein, 1991; Leon, Hoffman, and Staeger, 1996).

It is also useful to define a lower limit on strength below which the connection may be treated as a simple connection. Connections that transmit less than 20 percent of the fully plastic moment of the beam at a rotation of 0.02 radian may be considered to have no flexural strength for design. However, it should be recognized that the aggregate strength of many weak connections can be important when compared to that of a few strong connections (FEMA, 1997).

In Figure C-B3.3, the grey dot M_n indicates the maximum strength and the associated rotation θ_n. The open dot θ_u is the maximum rotation capacity. Note that it is possible for an FR connection to have a strength less than the strength of the beam. It is also possible for a PR connection to have a strength greater than the strength of the beam.

The strength of the connection must be adequate to resist the moment demands implied by the design loads.

Connection Ductility. If the connection strength substantially exceeds the fully plastic moment of the beam, then the ductility of the structural system is controlled by the beam and the connection can be considered elastic. If the connection strength only marginally exceeds the fully plastic moment of the beam, then the connection may experience substantial inelastic deformation before the beam reaches its full strength. If the beam strength exceeds the connection strength,

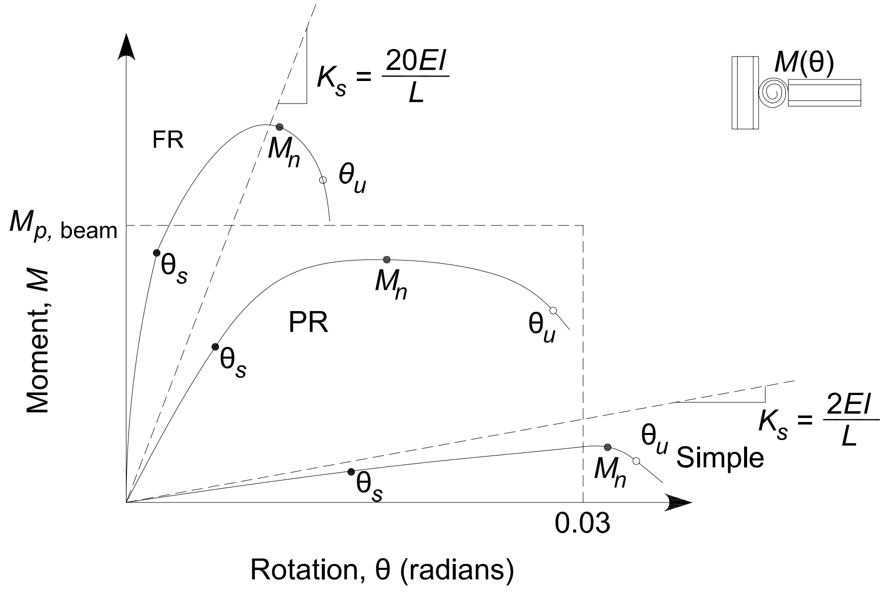


Fig. C-B3.3. Classification of moment-rotation response of fully restrained (FR), partially restrained (PR) and simple connections.

then deformations can concentrate in the connection. The ductility required of a connection will depend upon the particular application. For example, the ductility requirement for a braced frame in a nonseismic area will generally be less than the ductility required in a high seismic area. The rotation ductility requirements for seismic design depend upon the structural system (AISC, 2002).

In Figure C-B3.2, the rotation capacity, q_u, can be defined as the value of the connection rotation at the point where either (a) the resisting strength of the connection has dropped to $0.8M_n$ or (b) the connection has deformed beyond 0.03 radian. This second criterion is intended to apply to connections where there is no loss in strength until very large rotations occur. It is not prudent to rely on these large rotations in design.

The available rotation capacity, θ_u, should be compared with the rotation required at the strength limit state, as determined by an analysis that takes into account the nonlinear behavior of the connection. (Note that for design by ASD, the rotation required at the strength limit state should be assessed using analyses conducted at 1.6 times the ASD load combinations.) In the absence of an accurate analysis, a rotation capacity of 0.03 radian is considered adequate. This rotation is equal to the minimum beam-to-column connection capacity as specified in the seismic provisions for special moment frames (AISC, 2002). Many types of PR connections, such as top and seat-angle details, meet this criterion.

Structural Analysis and Design. When a connection is classified as PR the relevant response characteristics of the connection must be included in the analysis of the structure to determine the member and connection forces, displacements and the frame stability. Therefore, PR construction requires, first, that the moment-rotation characteristics of the connection be known and, second, that these characteristics be incorporated in the analysis and member design.

Typical moment-rotation curves for many PR connections are available from one of several databases [for example, Goverdhan (1983); Ang and Morris (1984); Nethercot (1985); and Kishi and Chen (1986)]. Care should be exercised when utilizing tabulated moment-rotation curves not to extrapolate to sizes or conditions beyond those used to develop the database since other failure modes may control (ASCE Task Committee on Effective Length, 1997). When the connections to be modeled do not fall within the range of the databases, it may be possible to determine the response characteristics from tests, simple component modeling, or finite element studies (FEMA, 1995). Examples of procedures to model connection behavior are given in the literature (Bjorhovde, Brozzetti, and Colson, 1988; Chen and Lui, 1991; Bjorhovde, Colson, Haaijer, and Stark, 1992; Lorenz and others, 1993; Chen and Toma, 1994; Chen, Goto, and Liew, 1995; Bjorhovde, Colson, and Zandonini, 1996; Leon, Hoffman, and Staeger, 1996; Leon and Easterling, 2002).

The degree of sophistication of the analysis depends on the problem at hand. Usually, design for PR construction requires separate analyses for the serviceability and strength limit states. For serviceability, an analysis using linear springs with a stiffness given by K_S (see Figure C-B3.2) is sufficient if the resistance demanded of the connection is well below the strength. When subjected to strength load combinations, a more careful procedure is needed so that the characteristics assumed in the analysis are consistent with those of the connection response. The response is especially nonlinear as the applied moment approaches the connection strength. In particular, the effect of the connection nonlinearity on second-order moments and other stability checks need to be considered (ASCE Task Committee on Effective Length, 1997).

7. Design for Serviceability

Section B3.7 provides the charging language for Chapter L on design for serviceability.

8. Design for Ponding

As used in this Specification, ponding refers to the retention of water due solely to the deflection of flat roof framing. The amount of this water is dependent on the flexibility of the framing. Lacking sufficient framing stiffness, the accumulated weight of the water can result in the collapse of the roof. The problem becomes catastrophic when more water causes more deflection, resulting in more room for more water until the roof collapses. Detailed provisions for determining ponding stability and strength are given in Appendix 2.

9. Design for Fatigue

Section B3.9 provides the charging language for Appendix 3 on design for fatigue.

10. Design for Fire Conditions

Section B3.10 provides the charging language for Appendix 4 on structural design for fire resistance. Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. It is anticipated that the basis will be ASCE/SFPE Standard 28 (ASCE, 1999), ASTM Standard E119 (ASTM, 2000), and similar documents.

11. Design for Corrosion Effects

Steel members may deteriorate in particular service environments. This deterioration may appear either in external corrosion, which would be visible upon inspection, or in undetected changes that would reduce its strength. The designer should recognize these problems by either factoring a specific amount of tolerance for damage into the design or providing adequate protection systems (for example, coatings, cathodic protection) and/or planned maintenance programs so that such problems do not occur.

Because the interior of an HSS is difficult to inspect, some concern has been expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection. Corrosion occurs in the presence of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS exposed to weather.

In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where conservative practice would recommend an internal protective coating include: (1) open HSS where changes in the air volume by ventilation or direct flow of water is possible; and (2) open HSS subject to a temperature gradient that would cause condensation.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to keep water from remaining in the HSS during or after construction, since the expansion caused by freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

12. Design Wall Thickness for HSS

ASTM A500 tolerances allow for a wall thickness that is not greater than ± 10 percent of the nominal value. Because the plate and strip from which electric-resistance-welded (ERW) HSS are made are produced to a much smaller thickness tolerance, manufacturers in the United States consistently produce ERW HSS with a wall thickness that is near the lower-bound wall thickness limit. Consequently, AISC and the Steel Tube Institute of North America (STI) recommend that 0.93 times the nominal wall thickness be used for calculations involving engineering design properties of ERW HSS. This results in a weight (mass) variation that is similar to that found in other structural shapes. Submerged-arc-welded (SAW) HSS are produced with a wall thickness that is near the nominal thickness and require no such reduction. The design wall thickness and section properties based upon this thickness have been tabulated in AISC and STI publications since 1997.

B4. CLASSIFICATION OF SECTIONS FOR LOCAL BUCKLING

For the purposes of this Specification, steel sections are divided into compact sections, noncompact sections and slender-element sections. Compact sections are capable of developing a fully plastic stress distribution and they possess a rotation capacity of approximately 3 before the onset of local buckling (Yura, Galambos, and Ravindra, 1978). Noncompact sections can develop partial yielding in compression elements before local buckling occurs, but will not resist inelastic local buckling at the strain levels required for a fully plastic stress distribution. Slender-element sections have one or more compression elements that will buckle elastically before the yield stress is achieved.

Limiting Width-Thickness Ratios. The dividing line between compact and noncompact sections is the limiting width-thickness ratio λ_p. For a section to be compact, all of its compression elements must have width-thickness ratios equal to or smaller than the limiting λ_p.

A second limiting width-thickness ratio is λ_r, representing the dividing line between noncompact sections and slender-element sections. As long as the width-thickness ratio of a compression element does not exceed the limiting value λ_r, elastic local buckling will not govern its strength. However, for those cases where the width-thickness ratios exceed λ_r, elastic buckling strength must be considered. Design procedures for such slender-element compression sections are given in Section E7 for members under pure axial compression, and in Sections F3.2, F5.3, F6.2, F7.2, F8.2, F9.3 and F10.3 for beams with a cross section that contains slender plate elements.

The values of the limiting ratios λ_p and λ_r specified in Table B4.1 are similar to those in the 1989 *Specification* (AISC, 1989) and Table 2.3.3.3 of Galambos (1978), except that $\lambda_p = 0.38\sqrt{E/F_y}$, limited in Galambos (1978) to indeterminate beams when moments are determined by elastic analysis and to determinate beams, was adopted for all conditions on the basis of Yura and others (1978).

For greater inelastic rotation capacities than provided by the limiting values λ_p given in Table B4.1, for structures in areas of high seismicity, see Section 8 and Table I-8-1 of the AISC *Seismic Provisions for Structural Steel Buildings* (AISC, 2005).

Flanges of Built-Up I-Shaped Sections. For built-up I-shaped sections under axial compression (Case 4 in Table B4.1), modifications have been made to the flange local buckling criterion to include web-flange interaction. The k_c in the λ_r *limit* and in Equations E7-7 through E7-9 is the same that is used for flexural members in Equations F3-2 and F5-9. Theory indicates that the web-flange interaction in axial compression is at least as severe as in flexure. Rolled shapes are excluded from this provision because there are no standard sections with proportions where the interaction would occur. In built-up sections where the interaction causes a reduction in the flange local buckling strength, it is likely that the web is also a thin stiffened element.

The k_c factor accounts for the interaction of flange and web local buckling demonstrated in experiments reported in Johnson (1985). The maximum limit of 0.76 corresponds to $F_{cr} = 0.69E/\lambda^2$ which was used as the local buckling strength in editions of both the ASD and LRFD Specifications. An $h/t_w = 27.5$ is required to reach $k_c = 0.76$. Fully fixed restraint for an unstiffened compression element corresponds to $k_c = 1.3$ while zero restraint gives $k_c = 0.42$. Because of web-flange interactions it is possible to get $k_c < 0.42$ from the new k_c formula. If $h/t_w > 5.70\sqrt{E/F_y}$ use $h/t_w = 5.70\sqrt{E/F_y}$ in the k_c equation, which corresponds to the 0.35 limit.

Webs in Flexure. New formulas for λ_p are presented in Case 11 in Table B4.1 for I-shaped beams with unequal flanges. These provisions are based on research reported in White (2003).

Rectangular HSS in Compression. The limits for rectangular HSS walls in uniform compression (Case 12 in Table B4.1) have been used in AISC Specifications since 1969. They are based on Winter (1968), where adjacent stiffened compression elements in box sections of uniform thickness were observed to provide negligible torsional restraint for one another along their corner edges. The λ_p limit for plastic analysis is adopted from *Limit States Design of Steel Structures* (CSA, 1994). The web slenderness limits are the same as those used for webs in wide-flange shapes.

Lower values of λ_p are specified for high-seismic design in the *Seismic Provisions for Structural Steel Buildings* based upon tests (Lui and Goel, 1987) that have shown that rectangular HSS braces subjected to reversed axial load fracture catastrophically under relatively few cycles if a local buckle forms. This was confirmed in tests (Sherman, 1995) where rectangular HSS braces sustained over 500 cycles when a local buckle did not form, even though general column buckling had occurred, but failed in less than 40 cycles when a local buckle developed. The seismic λ_p is based upon tests (Lui and Goel, 1987) of HSS that had a small enough *b*/*t* ratio so that braces performed satisfactorily for members with reasonable column slenderness. Filling the rectangular HSS with lean concrete (concrete mixed with a low proportion of cement) has been shown to effectively stiffen the HSS walls and improve cyclic performance.

Rectangular HSS in Flexure. A significant change from previous editions of the Specification is the compactness limit for webs in rectangular HSS flexural members (Case 13 in Table B4.1). The previously used value of $\lambda_p = 3.76\sqrt{E/F_y}$ was reduced to $\lambda_p = 2.42\sqrt{E/F_y}$. This change was introduced because tests reported in Wilkinson and Hancock (1998 and 2002) showed that HSS beams with geometries at the previous limiting compactness had hardly any rotation capacity available and were thus unable to deliver a target rotation capacity of 3.

Round HSS in Compression. The λ_r limit for round HSS in compression (Case 15 in Table B4.1) was first used in the 1978 *ASD Specification*. It was recommended in Schilling (1965) based upon research reported in Winter (1968). The same limit was also used to define a compact shape in bending in the 1978 *ASD Specification*. However, the limits for λ_p and λ_r were changed in the 1986 *LRFD Specification* based upon experimental research on round HSS in bending (Sherman, 1985;

Galambos, 1998). Excluding the use of round HSS with $D/t > 0.45E/F_y$ was also recommended in Schilling (1965).

Following the SSRC recommendations (Galambos, 1998) and the approach used for other shapes with slender compression elements, a Q factor is used for round sections to account for interaction between local and column buckling in Section E7.2(c). The Q factor is the ratio between the local buckling stress and the yield stress. The local buckling stress for the round section is taken from the AISI provisions based on *inelastic action* (Winter, 1970) and is based on tests conducted on fabricated and manufactured cylinders. Subsequent tests on fabricated cylinders (Galambos, 1998) confirm that this equation is conservative.

Round HSS in Flexure. The high shape factor for round hollow sections (Case 15 in Table B4.1) makes it impractical to use the same slenderness limits to define the regions of behavior for different types of loading. In Table B4.1, the values of λ_p for a compact shape that can achieve the plastic moment, and λ_r for bending, are based on an analysis of test data from several projects involving the bending of round HSS in a region of constant moment (Sherman and Tanavde, 1984; Galambos, 1998). The same analysis produced the equation for the inelastic moment capacity in Section F7. However, a more restrictive value of λ_p is required to prevent inelastic local buckling from limiting the plastic hinge rotation capacity needed to develop a mechanism in a round HSS (Sherman, 1976).

The values of λ_r for axial compression and for bending are both based on test data. The former value has been used in building specifications since 1968 (Winter, 1970). Section F8 also limits the D/t ratio for any round section to $0.45E/F_y$. Beyond this, the local buckling strength decreases rapidly, making it impractical to use these sections in building construction.

B5. FABRICATION, ERECTION AND QUALITY CONTROL

Section B5 provides the charging language for Chapter M on fabrication, erection and quality control.

B6. EVALUATION OF EXISTING STRUCTURES

Section B6 provides the charging language for Appendix 5 on the evaluation of existing structures.

CHAPTER C

STABILITY ANALYSIS AND DESIGN

Chapter C addresses the stability analysis and design requirements for steel buildings and related structures. The chapter has been reorganized from the previous Specifications into two parts: Section C1 outlines general requirements for stability and specific stability requirements for individual members (for example, beams, columns, braces) and for systems, including moment frames, braced frame and shear walls, gravity frame systems, and combined systems. Section C2 addresses the calculation of required strengths including the definition of acceptable analysis methods and specific constraints to be placed on the analysis and design procedures. A discussion of the effective length factor, K, the column buckling stress, F_e, and associated buckling analysis methods is provided at the end of the commentary chapter.

C1. STABILITY DESIGN REQUIREMENTS

1. General Requirements

The stability of structures must be considered from the standpoint of the structure as a whole, including not only the compression members, but also the beams, bracing systems and connections. Stability of individual components must also be provided. Considerable attention has been given in the technical literature to this subject, and various methods are available to provide stability (Galambos, 1998). In all approaches, the method of analysis and the equations for component strengths are inextricably interlinked. Traditionally, the effects of unavoidable geometric imperfections (within fabrication and erection tolerances) and distributed yielding at strength limit states (including residual stress effects) are addressed solely within member strength equations. Correspondingly, structural analysis is conducted using the nominal or undeformed structure geometry and elastic stiffness. This Specification addresses this traditional approach, termed the Effective Length Method in this commentary, as well as a new approach which is termed the Direct Analysis Method, addressed in Appendix 7. The Direct Analysis Method includes nominal geometric imperfection and stiffness reduction effects directly within the structural analysis. In either the Effective Length or the Direct Analysis Method, structural analysis by itself is not sufficient to provide for the stability of the structure as a whole. The overall stability of the structure as well as the stability of individual elements is provided for by the combined calculation of the required strengths by structural analysis and the satisfaction of the member and connection design provisions of this Specification.

In general, it is essential that an accurate second-order analysis of the structure be performed. The analysis should consider the influence of second-order effects (including P-Δ and P-δ effects as shown in Figure C-C1.1) and of flexural, shear and axial deformations. More rigorous analysis methods allow formulations of simpler limit state models. One such example can be found in Appendix 7, where the new Direct Analysis Method is presented as an alternative method to improve and simplify design for stability. In this case, the inclusion of nominal geometric imperfection and member stiffness reduction effects directly in the analysis allows the use of $K = 1.0$ in calculating the in-plane column strength, P_n, within the beam-column interaction equations of Chapter H. This simplification comes about because the Direct Analysis Method provides a better estimate of the true load effects within the structure. The Effective Length Method, in contrast, includes the above effects indirectly within the member strength equations.

2. Member Stability Design Requirements

Chapters E through I contain the necessary provisions for satisfying member stability (in other words, the available strengths) given the load effects obtained from structural analysis and given specific bracing conditions assumed in the calculation of the member strengths. Where beam and column members rely upon braces that are not part of the lateral load resisting system to define their unbraced length, the braces themselves must have sufficient strength and stiffness to control member movement at the brace points. Appendix 6 contains all the requirements for braces that were previously contained within Chapter C of the 1999 *LRFD Specification* (AISC, 2000b). Design requirements for braces that are part of the lateral load resisting system (that is, braces that are included within the analysis of the structure) are addressed within Chapter C.

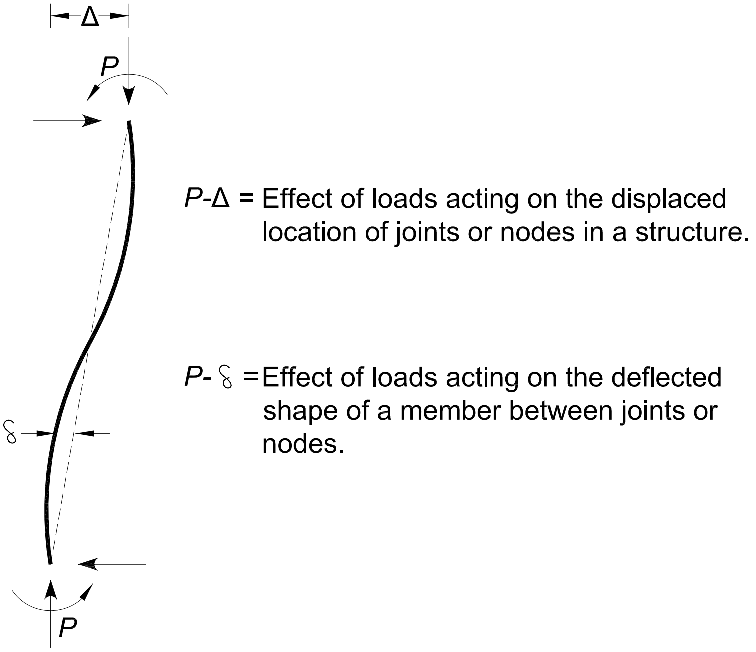


Fig. C-C1.1. P-Δ and P-δ effects in beam-columns.

3. System Stability Design Requirements

Lateral stability can be provided by braced frames, shear-wall systems, moment frames or any other comparable lateral load resisting systems. Where combined systems are used, it is important that consideration be given to the transfer of forces and load sharing between systems, and to the destabilizing effect of vertical load carrying elements not participating as part of the lateral load resisting system (for example, leaning columns).

3a. Braced-Frame and Shear-Wall Systems

Braced-frame systems are commonly analyzed and designed as vertically cantilevered pin-connected truss systems, ignoring any secondary moments within the system. The effective length factor, K, of components of the braced frame is normally taken to be 1.0, unless a smaller value is justified by structural analysis and the member and connection design is consistent with this assumption. Use of a K-factor less than 1.0 is discussed further at the end of this commentary chapter.

3b. Moment-Frame Systems

Moment-frame systems rely primarily upon the flexural stiffness of the connected beams and columns although the reduction in the stiffness due to shear deformations can be important and should be considered where column bays are short and/or members are deep. Except as noted in Section C2.2a(4), Section C2.2b and Appendix 7, the design of all columns and beam-columns must be based on an effective length, KL, greater than the actual length determined as specified in Section C2. The Direct Analysis Method in Appendix 7, as well as the provisions of Sections C2.2a(4) and C2.2b, provide the means for proportioning columns with $K = 1.0$.

3c. Gravity Framing Systems

Columns in gravity framing systems can be designed as pin-ended columns with $K = 1.0$. However, the destabilizing effect (P-Δ effect) of the gravity load on all such columns and the load transfer from these columns to the lateral load resisting system must be accounted for in the design of the lateral load resisting system. Methods for including this leaning column effect in the design of the lateral system are discussed in Commentary Section C2.

3d. Combined Systems

When combined systems are used, structural analysis must proportion the lateral loads to the various systems with due regard to the relative stiffness of each system and the load transfer path between them. Consideration must be given to the variation in stiffness inherent in concrete or masonry shear walls due to various degrees of cracking possible. This applies both to serviceability load combinations and strength load combinations. It is prudent for the designer to consider a range of possible stiffnesses, with due regard to shrinkage, creep and load history, in order to envelope the likely behavior and provide sufficient strength in all interconnecting elements between systems. Once the loads are determined on each system, the design must conform to all requirements for the respective systems.

C2. CALCULATION OF REQUIRED STRENGTHS

This Specification recognizes a variety of analysis and design procedures for assessing the response of lateral load resisting systems. These include the use of second-order inelastic and plastic methods with specially developed computer software, effective length factors in conjunction with second-order elastic analysis, the Direct Analysis Method, and simplified first-order elastic methods suitable for manual calculation. Accordingly, Section C2 addresses several general analysis approaches commonly used and defines certain constraints that must be placed on the analysis and design with each method so as to provide a safe design.

1. Methods of Second-Order Analysis

Some of the key differences between the 1999 *LRFD Specification* (AISC, 2000b) and this Specification involve requirements for minimum stiffness and strength of steel frames. The provisions in AISC (2000b) imposed the following two requirements on braced frames only:

(1) A minimum brace strength of

$$P_{br} = 0.004\Sigma P_u$$

(2) A minimum brace stiffness of

$$\beta_{br} = 2\Sigma P_u/(\phi L) \text{ where } \phi = 0.75$$

By substituting the minimum required brace stiffness, β_{br}, into the B_2 equation below [Equation C1-4 in AISC (2000b) where $\beta_{br} = \Sigma H/\Delta_{oh}$], it can be observed that the above minimum brace stiffness is equivalent to providing $B_2 \leq 1.6$. The minimum brace force, $P_{br} = 0.004\ \Sigma P_u$, is the force one would obtain in the brace by doing a first-order elastic analysis at the strength load level, including an initial out-of-plumbness of 0.002 times the story height, L, and assuming an amplification from second-order effects of 2.0. The amplification of 2.0 is determined using $\beta_{br} = 2\Sigma P_u/(\phi L)$ in the B_2 equation below, but without including the ϕ factor on stiffness.

$$B_2 = \frac{1}{1 - \dfrac{\Sigma P_u \Delta_{oh}}{\Sigma HL}} = \frac{1}{1 - \dfrac{\Sigma P_u}{\beta_{br} L}} \qquad \text{(C-C2-1)}$$

In contrast, this Specification imposes a minimum stiffness on all frames by application of a B_2 limit of 1.5 unless the more accurate Direct Analysis Method of Appendix 7 is used. The Direct Analysis Method addresses the influence of nominal geometric imperfections (for example, out-of-plumbness) and stiffness reductions due to distributed yielding directly within the analysis, in which case the above stiffness and strength requirements are accounted for in a direct manner. Setting the B_2 equation above to 1.5 is equivalent to imposing a minimum frame stiffness of $\beta_{br} = 3\ \Sigma P_u/L$ which is 12 percent larger than in AISC (2000b) for braced systems. The 12 percent difference is a consequence of setting the B_2 limit at 1.5 for all frames designed without the use of the more accurate Direct Analysis Method. Additional discussion about upper limits on B_2 can be found in Appendix 7, Section 7.3.

In the development of this Specification, it was considered to require an additive notional load of $0.002\Sigma Y_i$ with all load combinations for all B_2 levels. However, $(\Sigma H + 0.002\Sigma Y_i)/\Sigma H$ is close to 1.0 for all of the lateral load combinations in SEI/ASCE 7 (ASCE, 2002), and for $B_2 \leq 1.5$, the additional internal forces caused by applying $0.002\Sigma Y_i$ in combination with the required lateral loadings are small and may be neglected. Therefore, $0.002\Sigma Y_i$ is required only as a minimum lateral load in the gravity load-only combinations within Section C2.2a. Conversely, for frames with $B_2 > 1.5$, the P-Δ effects associated with the amplified lateral deflections due to initial out-of-plumbness plus the additional amplified defections due to distributed yielding or other incidental causes can be significant at strength load levels. Therefore, for these stability-sensitive structures the Direct Analysis Method of Appendix 7 is required with the use of an additive notional lateral load of $N_i = 0.002\Sigma Y_i$.

1a. General Second-Order Elastic Analysis

Section C2.1a states that any second-order elastic analysis method that captures both the P-Δ and P-δ effects, when one or both are significant to the accurate determination of internal forces, may be used. The amplification of first-order analysis forces by the traditional B_1 and B_2 factors as defined in Section C2.1b is one method of conducting an approximate second-order elastic analysis. In addition, the section states that all flexural, shear and axial deformations that significantly affect the stability of the structure and its elements in general must be considered. Also, in the Direct Analysis Method, nominal geometric imperfections and member stiffness reduction due to residual stresses must be directly included in the analysis.

The Direct Analysis Method is more sensitive to the accuracy of the second-order elastic analysis than the Effective Length Method. The Direct Analysis method may be used in the analysis and design of all lateral load resisting systems. The Commentary to Appendix 7, Sections 7.1 and 7.3, contains specific guidelines on the requirements for rigorous second-order elastic analysis, and provides benchmark problems that may be used to determine the adequacy of a particular analysis method. Software programs being used in the analysis should be tested with these benchmark problems to check their accuracy and to understand their limitations. Also, it is essential for the designer to apply the specific constraints applicable to the analysis-design method being used.

It is important to recognize that traditional elastic analysis methods, even those that properly consider second-order effects, are based on the undeformed geometry and nominal member properties and stiffnesses. Initial imperfections in the structure, such as out-of-plumbness, fabrication tolerances, incidental patterned gravity loading, temperature gradients across the structure, foundation settlements, etc., as well as residual member stresses and general softening of the structure at the strength limit state, combine with the destabilizing effects of the vertical loads to increase the magnitude of load effects in the structure above those predicted by traditional analysis methods. This is particularly true for stability-sensitive structures containing large vertical loads with small lateral load requirements, leading to relatively low lateral load resistance. Limits on B_2 are placed on some of the analysis-design methods to limit the potential underestimation of load effects in stability-sensitive structures. Note that B_2 may be determined directly as the ratio of the second-order to the first-order lateral displacements at each story in the structure, $\Delta_{2nd\ order}/\Delta_{1st\ order}$, (the appropriate definition when a second-order analysis is performed), or as defined by Equation C2-3 (the appropriate definition when an amplified first-order analysis is performed). This underestimation of load effects is particularly important in the design of restraining girders of moment frames and braces in braced frames. Within the Effective Length Method, the in-plane column strength, P_n, accounts for the above effects by inclusion of the effective length factor and the use of the column strength curve of Section E3. However, the increases in the magnitude of the internal forces due to these effects are not accounted for within other member and connection design equations. The Direct Analysis Method in Appendix 7 overcomes these shortcomings in the traditional Effective Length Method. Therefore, it is recommended for use, particularly in stability-sensitive structures.

1b. Second-Order Analysis by Amplified First-Order Elastic Analysis

Section C2.1b addresses the traditional amplified first-order analysis method that has long been part of this Specification. It has been expanded for use in systems where axial load is predominant, such as braced frames and truss systems, as well as moment frames. Where properly applied, this method constitutes an acceptable elastic second-order analysis method.

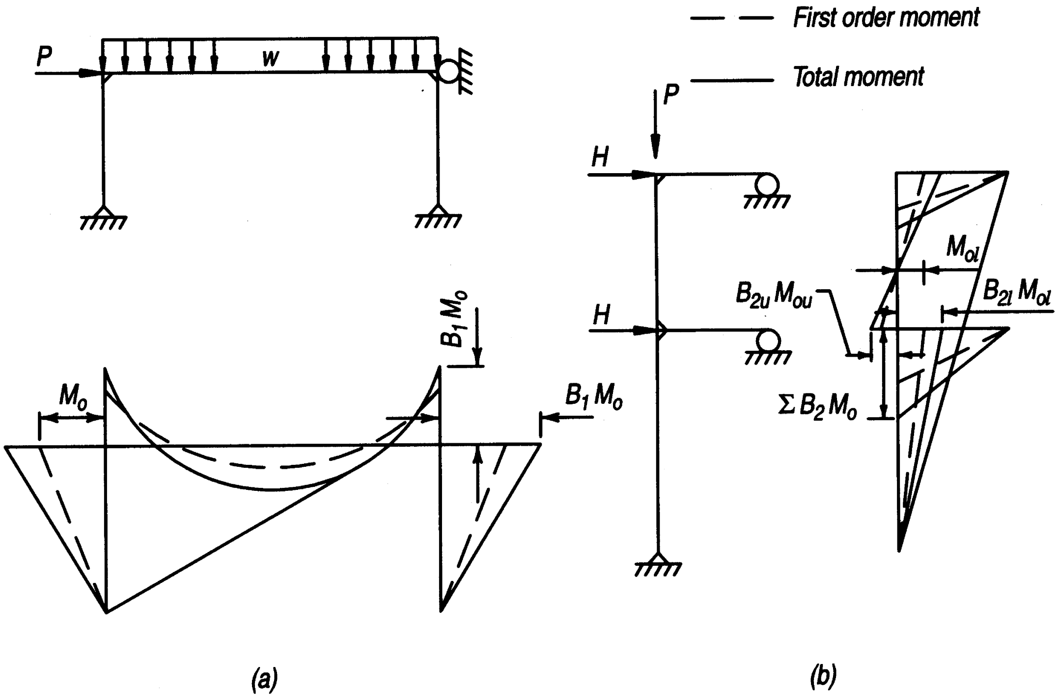
This first-order analysis method defines amplification factors B_1 and B_2 that are applied to the first-order forces so as to obtain an estimate of the second-order forces. In the general case, a member may have first-order load effects not associated with sidesway that are multiplied by B_1 and first-order load effects produced by sidesway that are multiplied by B_2. The factor B_1 is required to estimate the P-δ effects on the nonsway moments, M_{nt}, in axially loaded members, while the factor B_2 is required to estimate the P-Δ effect in frame components of braced, moment and/or combined framing systems. The P-Δ and P-δ effects are shown graphically in Figure C-C1.1 for a beam column. The effect of B_1 and B_2 amplification of moments is shown in Figure C-C2.1.

The factor B_2 applies only to internal forces associated with sidesway and is calculated for an entire story. In building frames designed to limit Δ_H/L to a predetermined value, the factor B_2 may be found in advance of designing individual members by using the target maximum limit on Δ_H/L within Equation C2-6b.

In determining B_2 and the second-order effects on the lateral load resisting system, it is important that Δ_H include not only the interstory displacement in the plane of the lateral load resisting system, but also any additional displacement in the floor or roof diaphragm or horizontal framing system that may increase the overturning effect of columns attached to and "leaning" against the horizontal system. Either the maximum displacement or a weighted average displacement, weighted in proportion to column load, should be considered.

Drift limits may also be set for design of various categories of buildings so that the effect of secondary bending is reduced (ATC, 1978; Kanchanalai and Lu, 1979). However, drift limits alone are not sufficient to allow stability effects to be neglected (LeMessurier, 1977).

Both types of first-order moments, M_{nt} and M_{lt}, may be induced by gravity loads. M_{nt} is defined as a moment developed in a member with frame sidesway prevented. M_{lt} is the moment developed within a member due to frame sidesway. If a significant restraining force is necessary to prevent sidesway of an unsymmetrical structure or an unsymmetrically loaded symmetrical structure, the moments induced by releasing the restraining force contribute to the M_{lt} moments. In most reasonably symmetric frames, this effect will be small. If the moment $B_2 M_{lt}$ is added algebraically to the $B_1 M_{nt}$ moment developed with sidesway prevented, as defined by Equation C2-1a, a reasonably accurate value of M_r results in most cases. A rigorous second-order elastic analysis is recommended for accurate determination of the frame internal forces when B_1 is larger than about 1.2. End moments produced in sidesway frames by lateral loads from wind or earthquake are always M_{lt} moments. Note that, in general, axial forces must also be amplified



(a) (b)

Fig. C-C2.1. Moment amplification.

according to Equation C2-1b for braced and moment frames, although the effect may be small in many low rise moment frames.

When first-order end moments in members subject to axial compression are magnified by B_1 and B_2 factors, equilibrium requires that they be balanced by moments in connected members (for example, see Figure C-C2.1). The associated second-order internal moments in the connected members can be calculated satisfactorily in most cases by amplifying the moments in all the members of the lateral load resisting system, in other words, the columns and the beams, by their corresponding B_1 and B_2 values. For beam members, the larger of the B_2 values from the story above or below is used. Connections shall also be designed to resist the magnified end moments. Alternatively, the difference between the magnified moment and the first-order moment in the column(s) at a given joint may be distributed to any other moment-resisting members attached to the compressed member (or members) in proportion to the relative stiffness of the uncompressed members. Minor imbalances may be neglected in the judgment of the engineer. This latter method is considerably more tedious than the above recommended method. Complex conditions, such as occur when there is significant magnification in several members meeting at a joint, may require an actual second-order elastic analysis rather than an amplified first-order analysis.

In braced and moment frames, P_n is governed by the maximum slenderness ratio regardless of the plane of bending, if the member is subject to significant biaxial bending, or if Section H1.3 is not utilized. Section H1.3 is an alternative approach for checking beam-column strength that provides for the separate checking of beam-column in-plane and out-of-plane stability in members predominantly subject to bending within the plane of the frame. However, P_{e1} and P_{e2} expressed by Equations C2-5 and C2-6a are always calculated using the slenderness ratio in the plane of bending. Thus, when flexure in a beam-column is about the strong axis only, two different values of slenderness ratio may be involved in the amplified first-order elastic and design calculations.

The value of $R_M = 0.85$ within Equation C2-6b is based on an approximate upper-bound influence of P-δ effects on the amplification of the sidesway displacements in practical moment frames (LeMessurier, 1977).

The second-order internal forces from separate structural analyses cannot normally be combined by superposition since second-order amplification depends, in a nonlinear fashion, on the total axial forces within the structure. Therefore, a separate second-order analysis must be conducted for each load combination considered in the design. The first-order internal forces, calculated prior to amplification within the amplified first-order elastic analysis procedure of Section C2.1b, may be superimposed to determine the total first-order internal forces.

When bending occurs about both the x- and the y-axes, the required flexural strength, calculated about each axis, is amplified by B_1 based on the value of C_m and P_{e1} in Equation C2-2 corresponding to the moment gradient in the beam-column and its slenderness ratio in the plane of bending. A similar amplification by B_2 in the required flexural strength must occur for ΣP_{e2} in Equation C2-3 corresponding to the in-plane response.

Equations C2-2 and C2-4 are used to approximate the maximum second-order moments in compression members with no relative joint translation and no transverse loads between the ends of the member. Figure C-C2.2a compares the approximation for C_m in Equation C2-4 to the exact theoretical solution for beam-columns subjected to applied end moments (Chen and Lui, 1987). This figure plots the approximate and analytical values of C_m versus the end-moment ratio M_1/M_2 for several levels of $P/P_e(P_e = P_{e1}$ with $K = 1)$. Figure C-C2.2b shows the corresponding approximate and analytical solutions for the maximum second-order elastic moment within the member, M_r, versus the axial load level, P/P_e, for several values of the end moment ratio M_1/M_2.

For beam-columns with transverse loadings, the second-order moment can be approximated by

$$C_m = 1 + \psi\left(\frac{\alpha P_r}{P_{e1}}\right) \qquad \text{(C-C2-2)}$$

for simply supported members

where

$\psi = \dfrac{\pi^2 \delta_o EI}{M_o L^2} - 1$

δ_o = maximum deflection due to transverse loading, in. (mm)

M_o = maximum first-order moment within the member due to the transverse loading, kip-in. (N-mm)

α = 1.0 (LRFD) or 1.6 (ASD)

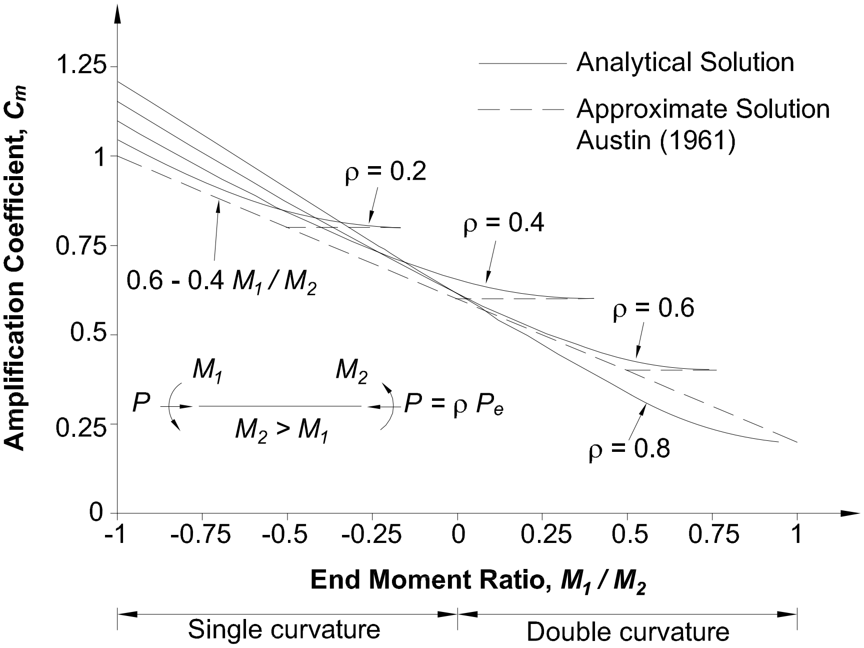


Fig. C-C2.2a. Equivalent moment factor C_m for beam-columns subjected to applied end moments.

For restrained ends, some limiting cases are given in Table C-C2.1 together with two cases of simply supported beam-columns (Iwankiw, 1984). These values of C_m are always used with the maximum moment in the member. For the restrained-end cases, the values of B_1 are most accurate if values of $K < 1.0$, corresponding to the member end conditions, are used in calculating P_{e1}. In lieu of using the equations above, $C_m = 1.0$ is used conservatively for all transversely loaded members. It can be shown that the use of $C_m = 0.85$ for members with restrained ends, specified in previous Specifications, can sometimes result in a significant under-estimation of the internal moments. Therefore, the use of $C_m = 1.0$ is recommended as a simple conservative approximation for all cases involving transversely loaded members.

2. Design Requirements

Section C2.2 contains requirements for two of the three methods of elastic analysis and design of lateral load resisting frames allowed by this Specification: (a) design by elastic second-order analysis; and (b) design by elastic first-order analysis. Conformance to all the constraints of these methods as specified in this section satisfies the requirements of Section C1.1. Appendix 7 addresses the third method of analysis and design called the Direct Analysis Method. Both methods listed in this section specify that the structure should be analyzed using the nominal geometry and the nominal elastic stiffnesses (*EI, EA*) for all members, which is the traditional approach. In order to limit potential errors in the load effects in the structure from these simplified analyses, it is necessary to limit the sidesway amplification, as represented by $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or equivalently, the

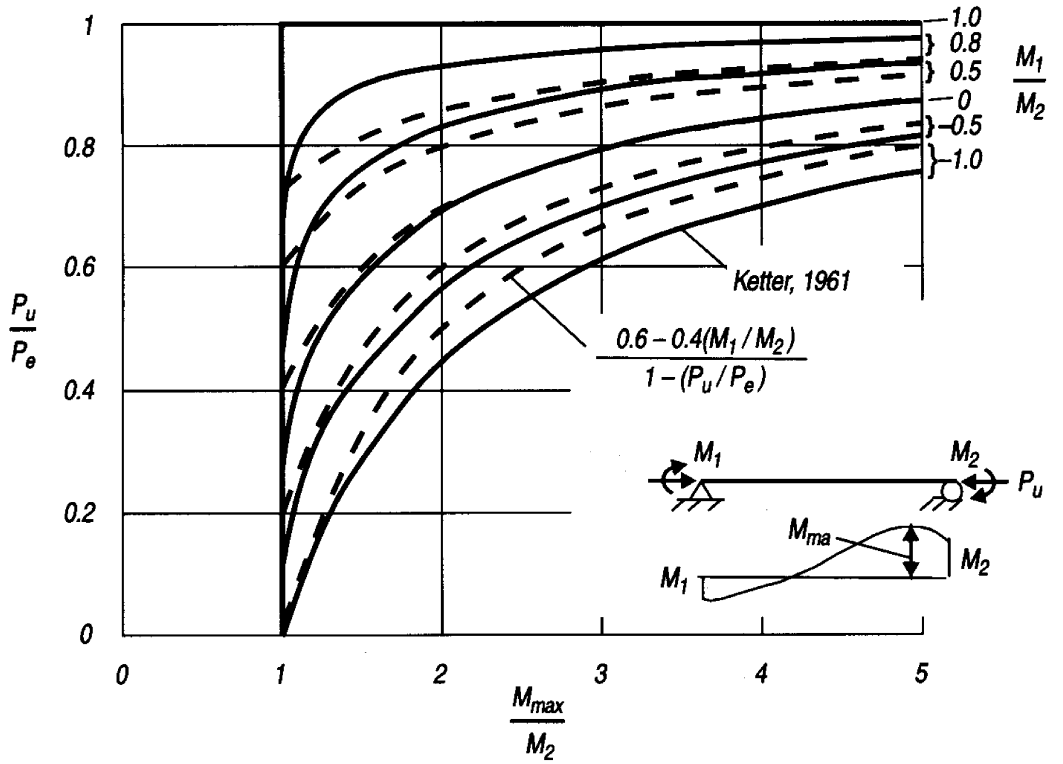
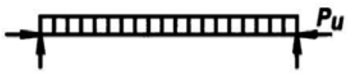
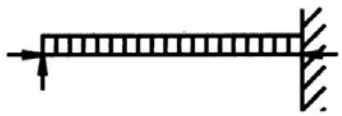
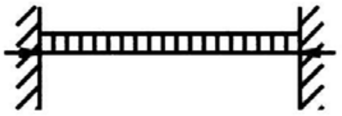
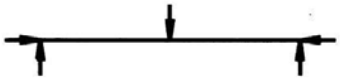
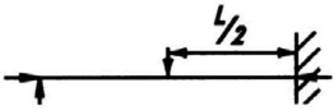
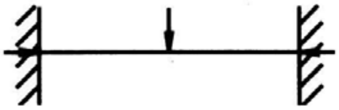


Fig. C-C2.2b. Second-order moments for beam-columns subjected to applied end moments.

TABLE C-C2.1
Amplification Factors ψ and C_m

Case	Ψ	C_m
	0	1.0
	–0.4	$1-0.4\frac{P_u}{P_{e1}}$
	–0.4	$1-0.4\frac{P_u}{P_{e1}}$
	–0.2	$1-0.2\frac{P_u}{P_{e1}}$
	–0.3	$1-0.3\frac{P_u}{P_{e1}}$
	–0.2	$1-0.2\frac{P_u}{P_{e1}}$

B_2 amplifier), in each story of the frame for all load combinations. A limit of 1.5 on $\Delta_{2nd\ order}/\Delta_{1st\ order}$ is specified for each of the methods addressed in Section C2.2a and C2.2b. Otherwise, the Direct Analysis Method in Appendix 7 is required. The Direct Analysis Method is applicable for any building frame, regardless of the sidesway amplification or B_2 value, and its use is encouraged.

It is important to note that the sidesway amplification or B_2 limits specified in Chapter C and Appendix 7 are based on Equation C2-3 which specifies a first-order elastic analysis using the nominal geometry and properties of the structure.

2a. Design by Second-Order Analysis

It is essential that the analysis of the frame be carried out at the strength limit state because of the nonlinearity associated with second-order effects. For design by the ASD method, this load level is estimated to be 1.6 times the ASD load combinations. This requirement is specified in clause (2).

Clause (3) in this section requires that, for all gravity load only combinations, a minimum lateral load of $0.002Y_i$ shall be applied at each level of the structure, where Y_i is the design gravity load acting on level i. Note that the load is to be applied independently in two orthogonal directions on the structure. Note also that the column strengths, P_n, in moment frames must be based on the effective buckling length, KL, or the column buckling stress, F_e, where either KL or F_e is determined from a sidesway buckling analysis of the structure. A detailed discussion of the K-factor, the column buckling stress, F_e, and associated sidesway buckling analysis methods is provided at the end of this commentary chapter.

In the special case where the sidesway amplification $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) ≤ 1.1, the frame design may be based on the use of $K = 1.0$ for the columns, as specified in clause (4). By limiting the sidesway amplification (or B_2 level) to a maximum value of 1.1, the resulting unconservative error is limited to a maximum of approximately 6 percent within the in-plane beam-column strength checks of Chapter H (White and Hajjar, 1997).

For all cases, braced frames may be designed on the basis of $K = 1.0$.

2b. Design by First-Order Analysis

This section provides a method for designing frames using a first-order elastic analysis with $K = 1.0$, provided the sidesway amplification $\Delta_{2nd\ order}/\Delta_{1st\ order} \leq 1.5$ (or $B_2 \leq 1.5$, where B_2 is determined as specified within the amplified first-order elastic analysis procedure of Section C2.1) and the required compressive strength of all members that are part of the lateral load resisting frame (other than truss members whose flexural stiffness is neglected in the analysis) have $\alpha P_r < 0.5P_y$. All load combinations must include an additional lateral load, N_i, applied in combination with other loads at each level of the structure specified by Equation C2-8. Note that the load is to be applied independently in two orthogonal directions on the structure. If drift occurs under gravity load, then the minimum load should be applied in the direction of the drift. This equation is derived from the Direct Analysis Method as shown in the commentary to Appendix 7. It is based on an assumed $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) value of 1.5. Initial out-of-plumbness does not need to be considered in the calculation of Δ. Equation C2-8 is based on the clause within Appendix 7 that permits the notional load to be applied as a minimum lateral load in the *gravity load only* combinations and not in combination with other lateral loads when $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2) ≤ 1.5. The minimum value of N_i of $0.0042Y_i$ is based on the assumption of a minimum first-order drift ratio due to any effects of $\Delta/L = 0.002$. Note that a target maximum drift ratio, corresponding to drifts under either the LRFD strength load combinations or 1.6 times the ASD strength load combinations, can be assumed at the start of design to determine the additional lateral load N_i. As long as that drift ratio is not exceeded at any strength load level, the design will be conservative.

The nonsway amplification of beam-column moments is addressed within the procedure specified in this section by applying the B_1 amplifier of Section C2.1 conservatively to the total member moments. In many cases involving beam-columns not subject to transverse loading between supports in the plane of bending, $B_1 = 1.0$.

Further explanation of this first-order design procedure is provided at the end of Appendix 7.

Determination of Effective Length Factor, *K*, or the Column Buckling Stress, F_e

There are two uses for the effective length factor, K, within the Specification:

(1) *Amplified first-order analysis.* K is used in the determination of the elastic buckling load, P_{e1}, for a member, or ΣP_{e2} for a building story, for calculation of the corresponding amplification factors B_1 and B_2 within the amplified first-order elastic analysis procedure of Section C2.1b; and

(2) *Column flexural buckling strength, P_n.* K is used in the determination of the column flexural buckling strength, P_n, from Chapter E, which may be based either on elastic or inelastic buckling analysis.

Each of these uses is discussed in detail below. The section begins, however, with a discussion of some background on the effective length factor, K, and some traditional approaches to determine K, most notably from the alignment charts.

Traditional Approaches to Calculating K—The Alignment Charts. A wide range of methods have been suggested in the engineering literature for the calculation of column effective length factors, K (Kavanagh, 1962; Johnston, 1976; LeMessurier, 1977; ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a). These range from simple idealizations of single columns such as shown in Table C-C2.2 to complex buckling solutions for specific frames and loading conditions. In some types of frames, K-factors are easily estimated or calculated and they serve as a convenient tool for stability design. In other types of structures, the determination of accurate K-factors is tedious by hand procedures, and system stability may be assessed more effectively without the consideration of member K values at all. This latter approach is addressed in more detail later in this section.

The most common method for determining K is with the use of the alignment charts, also commonly referred to as the nomographs, shown in Figure C-C2.3 for frames with sidesway inhibited and Figure C-C2.4 for frames with sidesway uninhibited. (Kavanagh, 1962) The appropriate subassemblages upon which the charts are based are shown in the figure, along with the alignment chart. The alignment charts are based on assumptions of idealized conditions which seldom exist in real structures. These assumptions are as follows:

1. Behavior is purely elastic
2. All members have constant cross section.
3. All joints are rigid.
4. For columns in frames with sidesway inhibited, rotations at opposite ends of the restraining beams are equal in magnitude and opposite in direction, producing single curvature bending.
5. For columns in frames with sidesway uninhibited, rotations at opposite ends of the restraining beams are equal in magnitude and direction, producing reverse curvature bending.

TABLE C-C2.2
Approximate Values of Effective Length Factor, *K*

Buckled shape of column is shown by dashed line.	(a)	(b)	(c)	(d)	(e)	(f)
						
Theoretical *K* value	0.5	0.7	1.0	1.0	2.0	2.0
Recommended design value when ideal conditions are approximated	0.65	0.80	1.2	1.0	2.10	2.0
End condition code	*Rotation fixed and translation fixed* *Rotation free and translation fixed* *Rotation fixed and translation free* *Rotation free and translation free*					

6. The stiffness parameter $L\sqrt{P/EI}$ of all columns is equal.
7. Joint restraint is distributed to the column above and below the joint in proportion to EI/L for the two columns.
8. All columns buckle simultaneously.
9. No significant axial compression force exists in the girders.

The alignment chart for sidesway inhibited frames shown in Figure C-C2.3 is based on the following equation:

$$\frac{G_A G_B}{4}(\pi/K)^2+\left(\frac{G_A+G_B}{2}\right)\left(1-\frac{\pi/K}{\tan(\pi/K)}\right)+\frac{2\tan(\pi/2K)}{(\pi/K)}-1=0$$

The alignment chart for sidesway uninhibited frames shown in Figure C-C2.4 is based on the following equation:

$$\frac{G_A G_B(\pi/K)^2-36}{6(G_A+G_B)}-\frac{(\pi/K)}{\tan(\pi/K)}=0$$

where

$$G=\frac{\Sigma(E_c I_c/L_c)}{\Sigma\left(E_g I_g/L_g\right)}=\frac{\Sigma(EI/L)_c}{\Sigma(EI/L)_g}$$

The subscripts *A* and *B* refer to the joints at the ends of the column being considered. The symbol Σ indicates a summation of all members rigidly connected to that joint and lying in the plane in which buckling of the column is being considered. E_c is the modulus of the column, I_c is the moment of inertia of the column, and L_c is the unsupported length of the column. E_g is the modulus of the girder, I_g is the moment of inertia of the girder, and L_g is the unsupported length of the girder or other restraining member. I_c and I_g are taken about axes perpendicular to the plane of buckling being considered. The alignment chart is valid for different materials if an appropriate effective rigidity, EI, is used in the calculation of G.

For column ends supported by, but not rigidly connected to, a footing or foundation, G is theoretically infinity but unless designed as a true friction-free pin, may be taken as 10 for practical designs. If the column end is rigidly attached to a properly designed footing, G may be taken as 1.0. Smaller values may be used if justified by analysis.

Theoretical K values obtained from the alignment charts for various idealized end conditions, rotation fixed or free and translation fixed or free, are shown in Table C-C2.2 along with practical recommendations for use in actual design.

It is important to remember that the alignment charts are based on the assumptions of idealized conditions previously discussed and that these conditions seldom exist

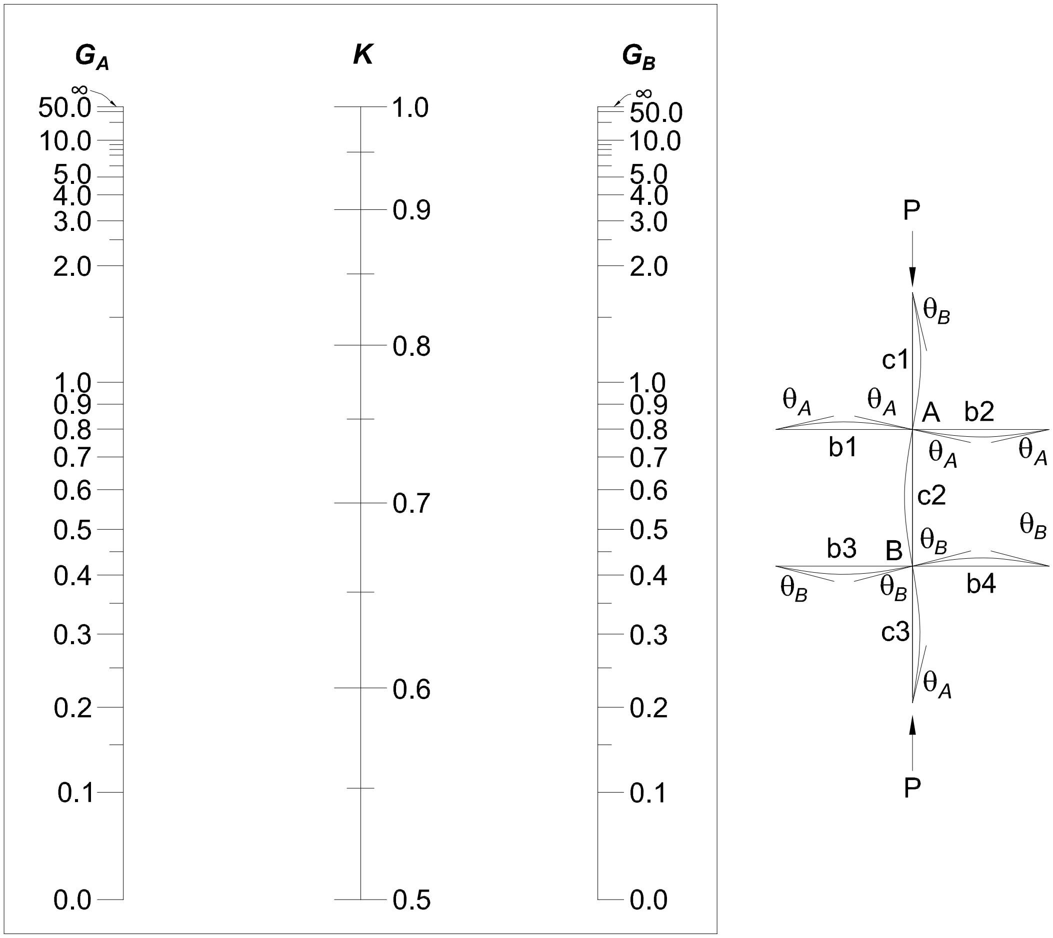


Fig. C-C2.3. Alignment chart—sidesway inhibited (braced frame).

in real structures. Therefore, adjustments are required when these assumptions are violated and the alignment charts are still to be used. Adjustments for common design conditions that apply to both sidesway conditions are:

1. To account for inelasticity in columns, replace $(E_c I_c)$ with $\tau_a(E_c I_c)$ for all columns in the expression for G_A and G_B. The stiffness reduction factor, τ_a, is discussed later in this section.
2. For girders containing significant axial load, multiply the $(EI/L)_g$ by the factor $(1-Q/Q_{cr})$ where Q is the axial load in the girder and Q_{cr} is the in-plane buckling load of the girder based on $K = 1.0$.

For sidesway inhibited frames, these adjustments for different beam end conditions may be made:

1. If the far end of a girder is fixed, multiply the $(EI/L)_g$ of the member by 2.0.
2. If the far end of the girder is pinned, multiply the $(EI/L)_g$ of the member by 1.5.

For sidesway uninhibited frames and girders with different boundary conditions, the modified girder length, L'_g, should be used in place of the actual girder length,

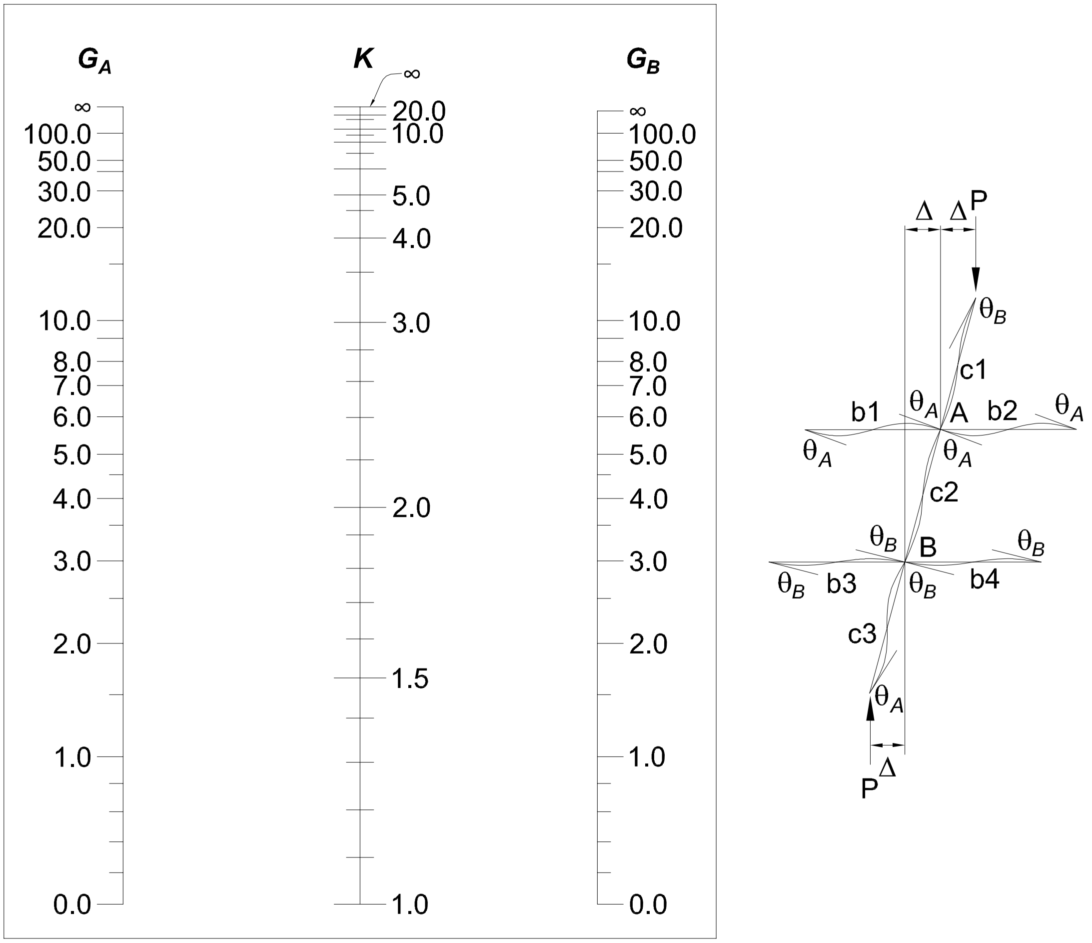


Fig. C-C2.4. Alignment chart—sidesway uninhibited (moment frame).

where

$$L'_g = L_g(2 - M_F/M_N)$$

M_F is the far end girder moment and M_N is the near end girder moment from a first-order lateral analysis of the frame. The ratio of the two moments is positive if the girder is in reverse curvature. If M_F/M_N is more than 2.0, then L'_g becomes negative, in which case G is negative and the alignment chart equation must be used.

1. If the far end of a girder is fixed, multiply the $(EI/L)_g$ of the member by $^2/_3$.
2. If the far end of the girder is pinned, multiply the $(EI/L)_g$ of the member by 0.5.

One important assumption in the development of the alignment charts is that all beam-column connections are fully restrained (FR connections). As seen above, when the far end of a beam does not have an FR connection that behaves as assumed, an adjustment must be made. When a beam connection at the column under consideration is a shear only connection—that is, there is no moment—then that beam can not participate in the restraint of the column and it cannot be considered in the $\Sigma(EI/L)_g$ term of the equation for G. Only FR connections can be used directly in the determination of G. PR connections with a documented moment-rotation response can be utilized, but the $(EI/L)_g$ of each beam must be adjusted to account for the connection flexibility. The ASCE Task Committee on Effective Length (ASCE, 1997) provides a detailed discussion of frame stability with PR connections.

Amplified First-Order Elastic Analysis (Section C2.1b). In this application of the effective length factor, K is used in the determination of the elastic critical buckling load, P_{e1}, for a member, or ΣP_{e2}, for a building story. These elastic critical buckling loads are then used for calculation of the corresponding amplification factors B_1 and B_2.

B_1 is used to estimate the P-δ effects on the nonsway moments, M_{nt}, in axially loaded members. K_1 is calculated in the plane of bending on the basis of no translation of the ends of the member and is normally set to 1.0, unless a smaller value is justified on the basis of analysis. There are also P-δ effects on the sway moments, M_{lt}, as explained previously in the discussion of Equation C2-6b.

B_2 is used to determine the P-Δ effect on the various components of moment, braced and/or combined framing systems. K_2 is calculated in the plane of bending through a sidesway buckling analysis. K_2 may be determined from the sidesway uninhibited alignment chart, Figure C-C2.4, without any correction for story buckling discussed later. ΣP_{e2} from the lateral load resisting columns with K_2 calculated in this way is an accurate estimate of the story elastic sidesway buckling strength. The contribution to the story sidesway buckling strength from leaning columns is zero, and therefore, these columns are not included in the summation in Equation C2-6a. However, the total story vertical load, including all columns in the story, is used for $\alpha\Sigma P_{nt}$ in Equation C2-3.

Since the amplified first-order elastic analysis involves the calculation of elastic buckling loads as a measure of frame and column stiffness, only elastic K factors are appropriate for this use.

Column Flexural Buckling Strength, P_n (Chapter E). In this application of effective length factors, K is used in the determination of the column flexural buckling strength, P_n, which may be based either in an elastic or inelastic buckling analysis.

The column elastic buckling stress, F_e, or the corresponding column axial force at incipient story elastic sidesway buckling, P_e, may be used directly in the calculation of the column flexural buckling strength, P_n. This is because the column strength equations of Chapter E (Equations E3-2 and E3-3) are a function of the ratio F_e/F_y. In fact, if the column axial stress at incipient buckling, F_e, is determined from any appropriate system buckling model, this value of F_e is all that is needed for the calculation of P_n.

The elastic column buckling stress, F_e, is given by Equation E3-4 as shown below:

$$F_e = \frac{\pi^2 E}{\left(\frac{KL}{r}\right)^2} \qquad \text{(C-C2-3)}$$

This equation uses the effective length factor, K, determined by a buckling analysis of a braced frame or a moment frame. F_e can also be obtained directly from a buckling analysis in which the column buckling load is P_e and

$$F_e = \frac{P_e}{A_g} \qquad \text{(C-C2-4)}$$

Other approaches for the determination of the effective length factor and the critical buckling load using simplified relationships have been presented in the literature. Several of these will be discussed later in this section.

Braced Frames: If $K < 1$ is used for the calculation of P_n in braced frames, the additional demands on stability bracing and the influence on the second-order moments in beams providing restraint to the columns must be considered. This Specification does not address the additional demands on bracing members from the use of $K < 1$. Generally, a rigorous second-order elastic analysis is necessary for calculation of the second-order moments in beams providing restraint to column members designed based on $K < 1$. Therefore, design using $K = 1$ is recommended, except in those special situations where the additional calculations are deemed justified.

Moment Frames: It is important to recognize that sidesway instability of a moment frame is a story phenomenon involving the sum of the sway resistances of each column in the story and the sum of the factored gravity loads in the columns in that story. No individual column in a story can buckle in a sidesway mode without all the columns in that story also buckling. If each column in a story of an moment frame is designed to support its own P and P-Δ moment such that the contribution of each column to the lateral stiffness, or to the story buckling load, is proportional to the axial load supported by the column, then all the columns will buckle simultaneously. Under this condition, there is no interaction among the columns in the story; column sway instability and frame instability occur at the same time. However, many common framing systems can be used that redistribute the story P-Δ effects to the columns in that story in proportion to their individual stiffnesses. This redistribution can be accomplished using such elements as floor diaphragms or horizontal trusses. In a moment frame that contains columns that contribute little or nothing to the sway stiffness of the story, such columns are referred to as leaning columns and they can be designed using $K = 1.0$. The other columns in the story must be designed to support the destabilizing P-Δ moments developed from the loads on these leaning columns. Similarly, the more highly loaded columns in a story will redistribute some of their P-Δ moments to the more lightly loaded columns. This phenomenon must be considered in the determination of K and F_e for all the columns in the story for the design of moment frames. The proper K-factor for calculation of P_n in moment frames, accounting for these effects, is denoted in the following by the symbol K_2.

Two methods for evaluating story frame stability, as measured by ΣP_{e2} for a story, are recognized: the story stiffness method (LeMessurier, 1976; LeMessurier, 1977) and the story buckling method (Yura, 1971). These are reflected in Chapter C with Equations C2-6b and C2-6a, respectively.

For the story stiffness method, K_2 is defined by

$$K_2 = \sqrt{\frac{\Sigma P_r}{(0.85 + 0.15R_L)\,P_r}\left(\frac{\pi^2 EI}{L^2}\right)\left(\frac{\Delta_H}{\Sigma HL}\right)} \geq \sqrt{\frac{\pi^2 EI}{L^2}\left(\frac{\Delta_H}{1.7HL}\right)} \qquad \text{(C-C2-5)}$$

This value of K_2 may be used in Equation C-C2-3 or directly in the equations of Chapter E. It is possible that certain columns, having only a small contribution to the lateral load resistance in the overall frame, will have a K_2 value less than 1.0 based on the term to the left of the inequality. The limit on the right-hand side is a minimum value for K_2 that accounts for the interaction between sidesway and nonsidesway buckling (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a). The term H is the shear in the column under consideration, produced by the lateral forces used to compute Δ_H.

It is important to note that this equation for K_2 is not appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b. It has been derived only for the determination of P_n defined in Chapter E.

Alternatively, Equation C-C2-5 can be reformulated to obtain the column buckling load for use in Equation C-C2-4 as

$$P_{e2} = \left(\frac{\Sigma HL}{\Delta_H}\right)\frac{P_r}{\Sigma P_r}(0.85 + 0.15R_L) \leq 1.7HL/\Delta_H \qquad \text{(C-C2-6)}$$

$$R_L = \frac{\Sigma P_r \textit{ leaning columns}}{\Sigma P_r \textit{ all columns}} \qquad \text{(C-C2-7)}$$

ΣP_r in Equations C-C2-5 and C-C2-6 includes all columns in the story, including any leaning columns, and P_r is for the column under consideration. The column load, P_{e2}, calculated from Equation C-C2-6 may be larger than $\pi^2 EI/L^2$ but may not be larger than the limit on the right hand side of this equation. R_L is the ratio of the vertical column load for all leaning columns in the story to the vertical load of all the columns in the story. This factor approaches 1.0 for systems with a large percentage of leaning columns. The purpose of R_L is to account for the debilitating influence of the P-δ effect on the sidesway stiffness of the columns of a story.

Note that ΣP_{e2} given by Equation C2-6b in the story stiffness method is expressed in terms of a building's story drift ratio Δ_H/L from a first-order lateral load analysis at a given applied lateral load level. In preliminary design, this may be taken in terms of a target maximum value for this drift ratio. This approach focuses the engineer's attention on the most fundamental stability requirement in building frames, providing adequate overall story stiffness in relation to the total vertical load, $\alpha \Sigma P_r$, supported by the story. The elastic story stiffness expressed in terms of the drift ratio and the total horizontal load acting on the story is $\Sigma H/(\Delta_H/L)$.

Story Buckling Method. For the story buckling method, K_2 is defined by

$$K_2 = \sqrt{\frac{\pi^2 EI/L^2}{P_r}\left(\frac{\Sigma P_r}{\Sigma \dfrac{\pi^2 EI}{(K_{n2}L)^2}}\right)} \geq \sqrt{\frac{5}{8}}K_{n2} \qquad \text{(C-C2-8)}$$

where K_{n2} is defined as the K value determined directly from the alignment chart in Figure C-C2.4. Again, the value for K_2 calculated from the above equation may be less than 1.0. The limit on the right hand side of this equation is a minimum value for K_2 that accounts for the interaction between sidesway and nonsidesway buckling (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a; Geschwindner, 2002; AISC-SSRC, 2003). It is again important to note that this equation for K_2 is *not* appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b. It has been derived only for the determination of P_n defined in Chapter E.

Alternatively, Equation C-C2-8 can be reformulated to obtain the column buckling load for use in Equation C-C2-4 as

$$P_{e2} = \left(\frac{P_r}{\Sigma P_r}\right)\Sigma\frac{\pi^2 EI}{(K_{n2}L)^2} \leq 1.6\frac{\pi^2 EI}{(K_{n2}L)^2} \qquad \text{(C-C2-9)}$$

The column load, P_{e2}, calculated from Equation C-C2-9, may be greater than $\pi^2 EI/L^2$ but may not be larger than the limit on the right-hand side of this equation. ΣP_r in Equations C-C2-8 and C-C2-9 includes all columns in the story, including any leaning columns, and P_r is for the column under consideration. K_{n2} in Equations C-C2-8 and C-C2-9 above is determined from the alignment chart in

Figure C-C2.4. Note also that the value of P_n, calculated using K_2 by either method cannot be taken greater than P_n, based on sidesway inhibited buckling. Other methods to calculate K_2 are given in previous editions of this commentary and are discussed elsewhere (ASCE Task Committee on Effective Length, 1997; White and Hajjar, 1997a; Geschwindner, 2002; AISC-SSRC, 2003).

Another simple formula for K_2 (LeMessurier, 1995), based only on the column end moments, is shown below:

$$K_2 = \left[1 + (1 - M_1/M_2)^4\right]\sqrt{1 + \frac{5}{6}\frac{\Sigma P_r \textit{ leaning columns}}{\Sigma P_r \textit{ nonleaning columns}}} \qquad \text{(C-C2-10)}$$

M_1 is the smaller and M_2 the larger end moment in the column. These moments are determined from a first-order analysis of the frame under wind load. Column inelasticity is considered in the derivation of this equation. The unconservative error in P_n using the above equation is less than 3 percent, as long as the following inequality is satisfied:

$$\left(\frac{\Sigma P_y \textit{ nonleaning columns}}{\Sigma HL/\Delta_H}\right)\left(\frac{\Sigma P_r \textit{ all columns}}{\Sigma P_r \textit{ nonleaning columns}}\right) \le 0.45 \qquad \text{(C-C2-11)}$$

As with the other approaches for determining K_2 in this section, this equation for K_2 is not appropriate for use in Equation C2-6a for determining ΣP_{e2} and B_2 in Section C2.1b.

Adjustments in K_2 for Column Inelasticity and Determination of P_n. Adjustments in the effective length factor, K_2, or the column buckling stress, F_e, in the calculation of the column strengths, P_n, can be made based on an inelastic buckling analysis of the frame and the inelasticity inherent in the column under the governing load combination (Yura, 1971; ASCE Task Committee on Effective Length, 1997). Columns loaded into the inelastic range of behavior can be viewed as having a tangent modulus, E_T, that is smaller than E. For such columns, $E_c = E_T$ in the equation for G, which usually gives smaller G values, and therefore, smaller K-factors than those based on elastic behavior. Note that it is usually conservative to base the calculation of P_n on elastic K-factors. For more accurate solutions, inelastic K-factors can be determined from the alignment chart method by using τ_a times E_c for E_c in the equation for G where $\tau_a = E_T/E$ is the column inelastic stiffness reduction factor. Depending on how it is calculated, τ_a may account for both a reduction in the stiffness of columns due to geometric imperfections and spread of plasticity from residual stresses under high compression loading:

(a) For $P_n/P_y \le 0.39$ (elastic):

$$\tau_a = 1.0$$

(b) For $P_n/P_y > 0.39$ (inelastic):

$$\tau_a = -2.724(P_n/P_y)\ln(P_n/P_y) \qquad \text{(C-C2-12)}$$

where P_y is the column squash load, $F_y A_g$, and P_n is the nominal column strength. It should be noted the determination of τ_a is in general an iterative process because

P_n (a function of F_e) is dependent upon τ_a. A conservative simplification that eliminates this iterative process is to use $\alpha P_r/\phi_c$ in place of P_n.

Column inelasticity can be considered in determining K_2 (Equations C-C2-5 and C-C2-8) or P_{e2} (Equations C-C2-6 and C- C2-9) for the story stiffness method and the story buckling method. In the story stiffness method, $\tau_a I_c$ can be substituted for I_c for all columns in the frame analysis used to determine Δ_H. In addition, $\tau_a I_c$ can be used in place of I in Equation C-C2-5. In the story buckling method, τ_a is used in the determination of K_{n2} from the alignment chart in Equations C-C2-8 and C-C2-9 and also in those same equations by replacing I_c with $\tau_a I_c$.

If the column inelastic buckling load (P_{e2} from Equations C-C2-6 and C- C2-9 above, modified for inelasticity as described in the above paragraph) is used to determine F_e from Equation C-C2-4 for use in Chapter E (Equations E3-2 and E3-3), then its value must be divided by τ_a as shown below:

$$F_e = \frac{P_{e2}\ (inelastic)}{\tau_a A_g} \tag{C-C2-13}$$

The term in the numerator of the above equation denotes the load in the column at incipient inelastic buckling (ASCE Task Committee on Effective Length, 1997). Alternatively, if an inelastic K_2 is determined using τ_a as described in the previous paragraph, this K factor may be substituted directly into Equation C-C2-3 for calculation of F_e.

Some Conclusions Regarding K. It is important to note that column design using K-factors can be tedious and confusing for complex building structures containing leaning columns and/or combined framing systems, particularly where column inelasticity is considered. This confusion can be avoided if the Direct Analysis Method of Appendix 7 is used, where P_n is always based on $K = 1.0$. Also, the first-order elastic design-analysis method of Section C2.2b is based on the Direct Analysis Method, and hence also uses $K = 1.0$ in the determination of P_n. Furthermore, under certain circumstances where B_2 is sufficiently low, a K-factor of 1.0 may be assumed in design by second-order analysis as specified in Section C2.2a (4). For frames that satisfy this clause, it is not appropriate to use $K = 1.0$ in the calculation of B_2 using Equations C2-6a and C2-3. The use of Equation C2-6b is recommended for the calculation of B_2 within this context.

CHAPTER D

DESIGN OF MEMBERS FOR TENSION

The provisions of Chapter D do not account for eccentricities between the lines of action of connected assemblies.

D1. SLENDERNESS LIMITATIONS

The advisory upper limit on slenderness in the User Note is based on professional judgment and practical considerations of economics, ease of handling and care required so as to minimize inadvertent damage during fabrication, transport, and erection. This slenderness limit is not essential to the structural integrity of tension members; it merely assures a degree of stiffness such that undesirable lateral movement ("slapping" or vibration) will be unlikely. Out-of-straightness within reasonable tolerances does not affect the strength of tension members. Applied tension tends to reduce, whereas compression tends to amplify, out-of-straightness.

For single angles, the radius of gyration about the z-axis produces the maximum l/r and, except for very unusual support conditions, the maximum Kl/r.

D2. TENSILE STRENGTH

Because of *strain hardening*, a ductile steel bar loaded in axial tension can resist without rupture a force greater than the product of its gross area and its specified minimum yield stress. However, excessive elongation of a tension member due to uncontrolled yielding of its gross area not only marks the limit of its usefulness but can precipitate failure of the structural system of which it is a part. On the other hand, depending upon the reduction of area and other mechanical properties of the steel, the member can fail by rupture of the net area at a load smaller than required to yield the gross area. Hence, general yielding of the gross area and rupture of the net area both constitute limit states.

The length of the member in the net area is generally negligible relative to the total length of the member. *Strain hardening* is easily reached in the vicinity of holes and yielding of the net area at fastener holes does not constitute a limit state of practical significance.

Except for HSS that are subjected to *cyclic load* reversals, there is no information that the factors governing the strength of HSS in tension differ from those for other structural shapes, and the provisions in Section D2 apply. Because the number of different end connection types that are practical for HSS is limited, the determination of the net effective area A_e can be simplified using the provisions in Chapter K.

D3. AREA DETERMINATION

1. Gross Area

For HSS, ASTM A500 tolerances allow for a wall thickness that is not greater than ± 10 percent under thickness; consequently the gross area for ASTM A500 HSS is to be computed using 93 percent of the nominal wall thickness. This reduction is included in the tabulated properties for these sections that are included in the AISC *Manual of Steel Construction* (AISC, 2005a).

2. Net Area

The critical net area is based on net width and load transfer at a particular chain. Because of possible damage around a hole during drilling or punching operations, $^1/_{16}$ in. (1.5 mm) is added to the nominal hole diameter when computing the critical net area.

3. Effective Net Area

Section D3.3 deals with the effect of shear lag, applicable to both welded and bolted tension members. The reduction coefficient U is applied to the net area A_n of bolted members and to the gross area A_g of welded members. As the length of the connection l is increased, the shear lag effect diminishes. This concept is expressed empirically by the equation for U. Using this expression to compute the effective area, the estimated strength of some 1,000 bolted and riveted connection test specimens, with few exceptions, correlated with observed test results within a scatterband of ±10 percent (Munse and Chesson, 1963). Newer research provides further justification for the current provisions (Easterling and Gonzales, 1993).

For any given profile and configuration of connected elements, $\bar{x}$ is the perpendicular distance from the connection plane, or face of the member, to the centroid of the member section resisting the connection force, as shown in Figure C-D3.1. The length l is a function of the number of rows of fasteners or the length of weld. The length l is illustrated as the distance, parallel to the line of force, between the first and last row of fasteners in a line for bolted connections. The number of bolts in a line, for the purpose of the determination of l, is determined by the line with the maximum number of bolts in the connection. For staggered bolts, the out-to-out dimension is used for l, as shown in Figure C-D3.2.

There is insufficient data for establishing a value of U if all lines have only one bolt, but it is probably conservative to use A_e equal to the net area of the connected element. The limit states of block shear (Section J4.3) and bearing (Section J3.10), which must be checked, will probably control the design.

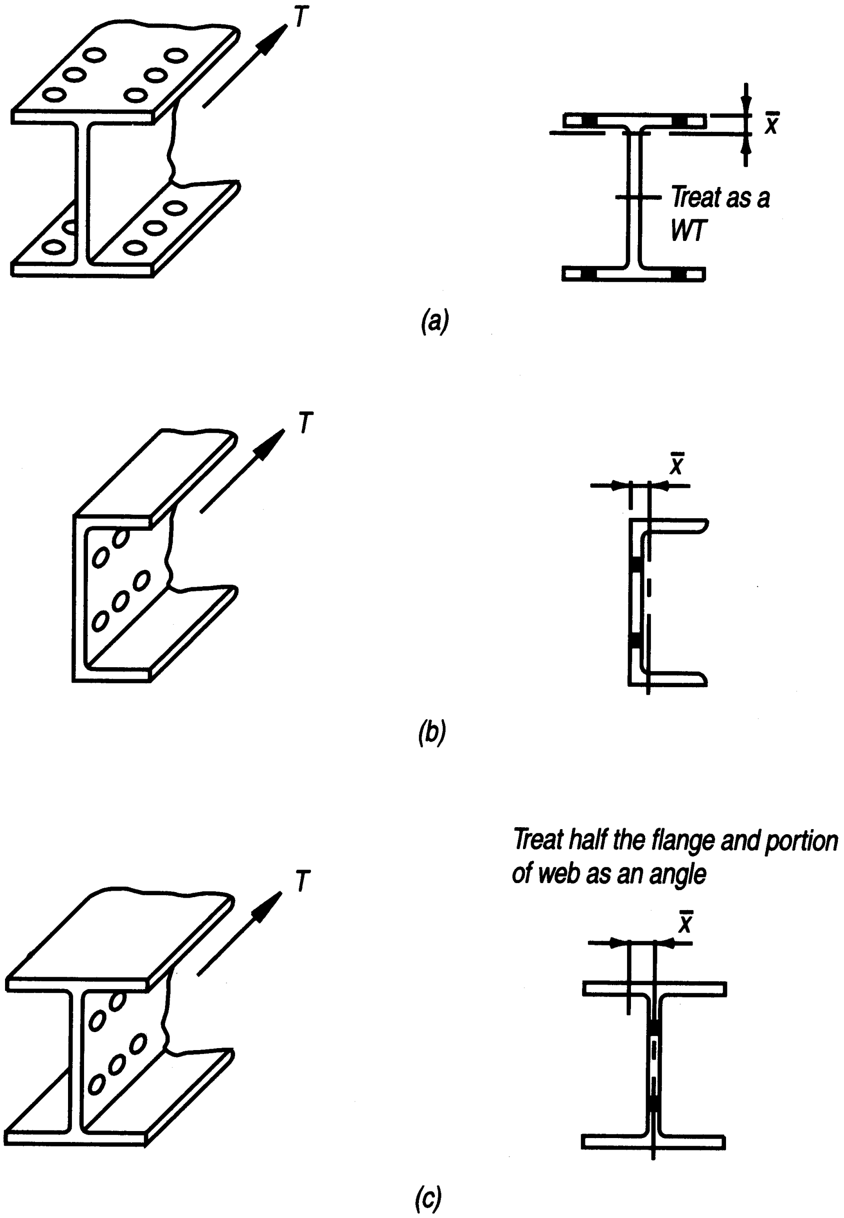


Fig. C-D3.1. Determination of $\bar{x}$ for U.

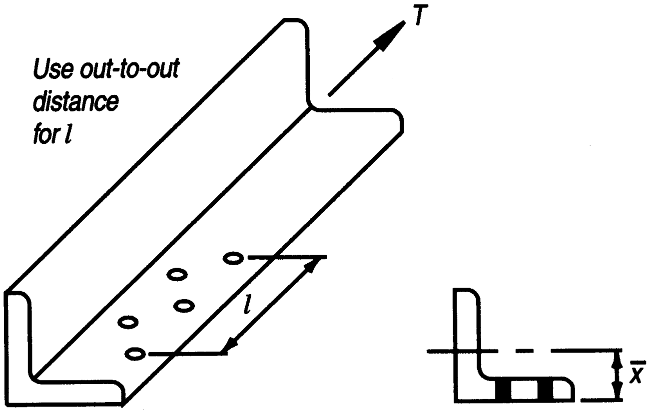


Fig. C-D3.2. Determination of l for U for bolted connections with staggered holes.

Significant eccentricity may exist within the connection if U is less than 0.6. For values of U less than 0.6 the connection may be used only if the provisions for members subject to combined bending and axial force are satisfied in the design of the member.

For welded connections, l is the length of the weld parallel to the line of force as shown in Figure C-D3.3 for longitudinal and longitudinal plus transverse welds.

End connections for HSS in tension are commonly made by welding around the perimeter of the HSS; in this case, there is no shear lag or reduction in the gross area. Alternatively, an end connection with gusset plates can be used. Single gusset plates may be welded in longitudinal slots that are located at the centerline of the cross section. Welding around the end of the gusset plate may be omitted for statically loaded connections to prevent possible *undercutting* of the gusset and having to bridge the gap at the end of the slot. In such cases, the net area at the end of the slot is the critical area as illustrated in Figure C-D3.4. Alternatively, a

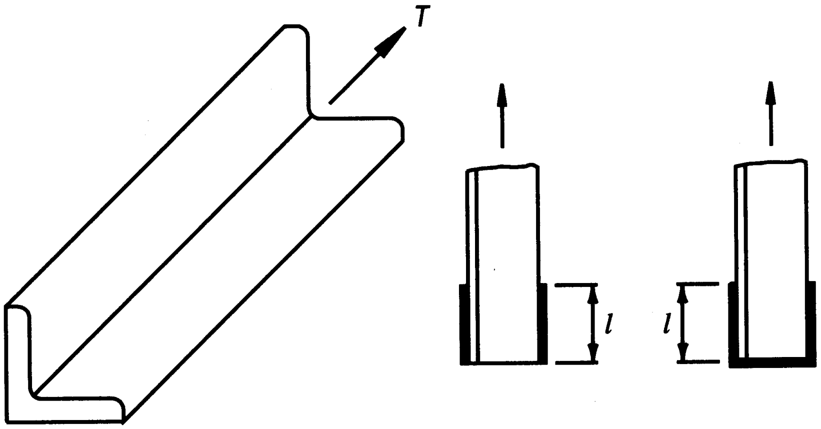


Fig. C-D3.3. Determination of l for U for connections with longitudinal and transverse welds.

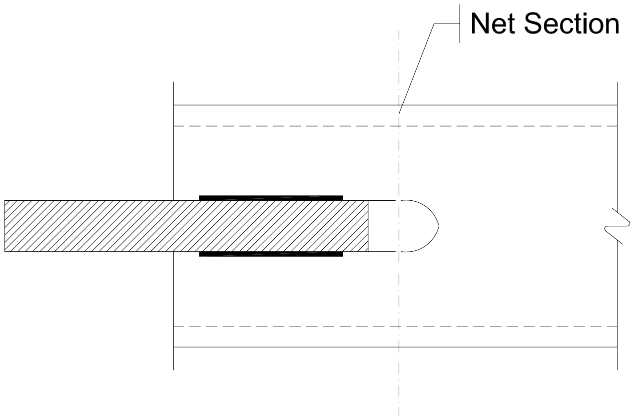


Fig. C-D3.4. Net area through slot for single gusset plate.

pair of gusset plates can be welded to opposite sides of a rectangular HSS with flare bevel groove welds with no reduction in the gross area.

For end connections with gusset plates, the general provisions for shear lag in Case 2 of Table D3.1 can be simplified and the connection eccentricity $\bar{x}$ can be explicitly defined as in Cases 5 and 6. In Cases 5 and 6 it is implied that the weld length, l, should not be less than the depth of the HSS. This is consistent with the weld length requirements in Case 4. In Case 5, the use of $U = 1$ when $l \geq 1.3D$ is based on research (Cheng and Kulak, 2000) that shows that fracture occurs only in short connections and that, in long connections, the round HSS tension member necks within its length and failure is by member yielding and eventual fracture.

The shear lag factors given in Cases 7 and 8 of Table D3.1 were located in the commentary of the 1999 *LRFD Specification* (AISC, 2000b) and are now given as alternate U values to the value determined from $1 - \bar{x}/l$ given for Case 2 in Table D3.1. It is permissible to use the larger of the two values.

D4. BUILT-UP MEMBERS

Although not commonly used, built-up member configurations using lacing, tie plates and perforated cover plates are permitted by this Specification. The length and thickness of tie plates are limited by the distance between the lines of fasteners, h, which may be either bolts or welds.

D5. PIN-CONNECTED MEMBERS

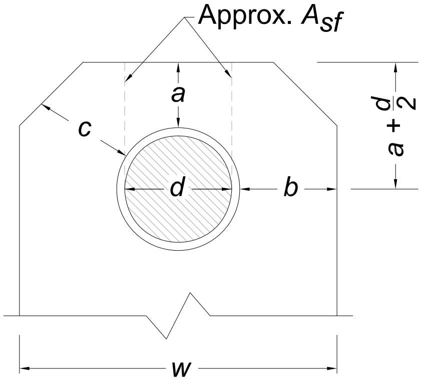
Pin-connected members are occasionally used as tension members with very large dead loads. Pin-connected members are not recommended when there is sufficient variation in live loading to cause wearing of the pins in the holes. The dimensional requirements presented in Specification Section D5.2 must be met to provide for the proper functioning of the pin.

1. Tensile Strength

The tensile strength requirements for pin-connected members use the same ϕ and Ω values as elsewhere in this Specification for similar limit states. However, the definitions of effective net area for tension and shear are different, as shown in Figure C-D5.1.

2. Dimensional Requirements

Dimensional requirements for pin-connected members are illustrated in Figure C-D5.1.



Dimensional Requirements

1. $a \geq 4/3\ b_{eff}$
2. $w \geq 2b_{eff} + d$
3. $c \geq a$

where

$$b_{eff} = 2t + 0.625 \text{ in. } (16 \text{ mm}) \leq b$$

Fig. C-D5.1. Dimensional requirements for pin-connected members.

D6. EYEBARS

Forged eyebars have generally been replaced by pin-connected plates or eyebars thermally cut from plates. Provisions for the proportioning of eyebars contained in this Specification are based upon standards evolved from long experience with forged eyebars. Through extensive destructive testing, eyebars have been found to provide balanced designs when they are thermally cut instead of forged. The more conservative rules for pin-connected members of nonuniform cross section and for members not having enlarged "circular" heads are likewise based on the results of experimental research (Johnston, 1939).

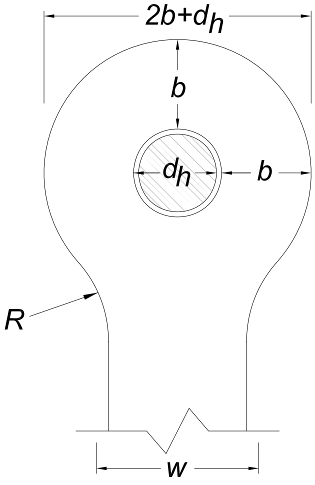
Stockier proportions are required for eyebars fabricated from steel having a yield stress greater than 70 ksi (485 MPa) to eliminate any possibility of their "dishing" under the higher design stress.

1. Tensile Strength

The tensile strength of eyebars is determined as for general tension members, except that, for calculation purposes, the width of the body of the eyebar is limited to eight times its thickness.

2. Dimensional Requirements

Dimensional limitations for eyebars are illustrated in Figure C-D6.1.



Dimensional Requirements

$t \geq 1/2$ in. (13mm) (Exception is provided in D6.2)

$w \leq 8t$

$d \geq 7/8\,w$

$d_h \leq d + 1/32$ in. (1mm)

$R \geq d_h + 2b$

$2/3\,w \leq b \leq 3/4\,w$ (Upper limit is for calculation purposes only)

Fig. C-D6.1. Dimensional limitations for eyebars.

CHAPTER E

DESIGN OF MEMBERS FOR COMPRESSION

E1. GENERAL PROVISIONS

The basic column equations in Section E3 are based on a reasonable conversion of research data into strength equations (Tide, 1985; Tide, 2001). These equations are essentially the same as those in the three previous editions of the *LRFD Specification* (see the discussion in Commentary Section E3 for further discussion). The one significant difference between the previous *LRFD Specifications* and this Specification is that the resistance factor ϕ has been increased from 0.85 to 0.90. The reasons for this increase are the changes in industry practice since the original calibrations were performed in the 1970s.

In the original research on the probability-based strength of steel columns (Bjorhovde, 1972; Bjorhovde, 1978) three *column curves* were recommended. These three *column curve*s were the mean equations of data bands of columns of similar manufacture. For example, hot-formed and cold-formed heat treated HSS columns fell into the data band of highest strength [SSRC Column Category P1 in Galambos (1998), Chapter 3], while welded built-up wide-flange columns made from universal mill plates were included in the data band of lowest strength (SSRC Column Category P3). The largest group of data clustered around SSRC Column Category P2. Had the original *LRFD Specification* opted for using all three *column curves* for the respective column categories, probabilistic analysis would have resulted in a resistance factor equal to $\phi = 0.90$ (Galambos, 1983; Galambos, 1998). It was decided, however, to employ only one *column curve*, SSRC Column Category P2, for all column types. This resulted in a larger data spread and thus in a larger coefficient of variation, and so a resistance factor $\phi = 0.85$ was adopted for the column equations to achieve a reliability comparable to that of beams.

The single *column curve* and the resistance factor of 0.85 were selected by the AISC Committee on Specifications in 1981 when the first draft of the *LRFD Specification* was developed (AISC, 1986). Since then there have been a number of changes in industry practice: (1) welded built-up shapes are no longer manufactured from universal mill plates; and (2) the yield strength of steel has increased with the standard constructional steel (ASTM 992) having a nominal yield stress of 50 ksi (345 MPa). The spread of the yield stress, in other words, its coefficient of variation, has been reduced (Bartlett and others, 2003).

An examination of the SSRC *Column Curve* Selection Table [Figure 3.27 in Galambos (1998)] reveals that there is no longer any SSRC P3 *Column Curve* Category. It is now possible to conservatively use only the statistical data for SSRC Column

Category P2 for the probabilistic determination of the reliability of columns. The curves in Figures C-E1.1 and C-E1.2 show the variation of the reliability index β with the live-to-dead load ratio L/D in the range of 1 to 5 for LRFD with $\phi = 0.90$ and ASD with $\Omega = 1.67$, respectively, for $F_y = 50$ ksi (345 MPa). The reliability index does not fall below $\beta = 2.6$. This is comparable to the reliability of beams. The ASD method gives higher reliability in the lower L/D range than the LRFD method.

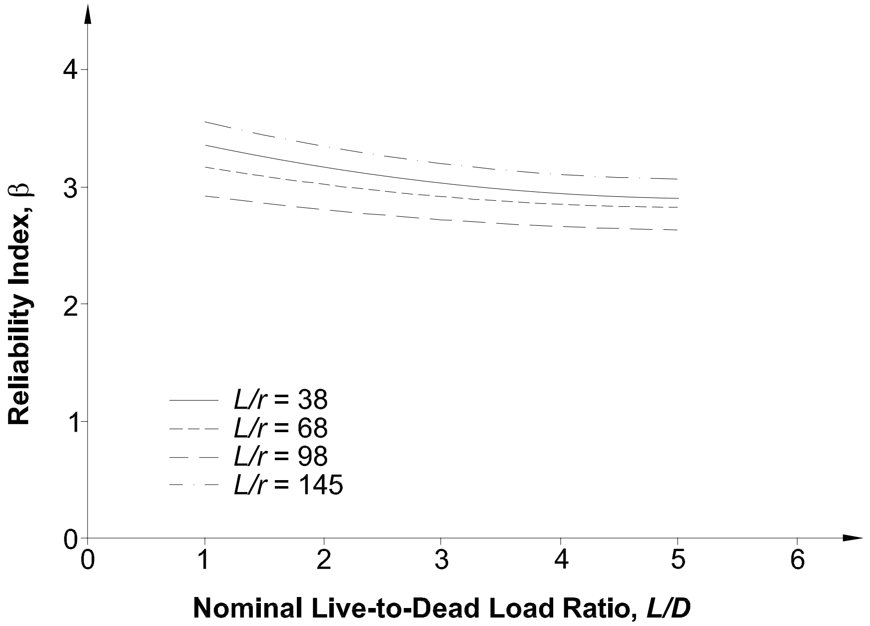


Fig. C-E1.1. Reliability of columns (LRFD).

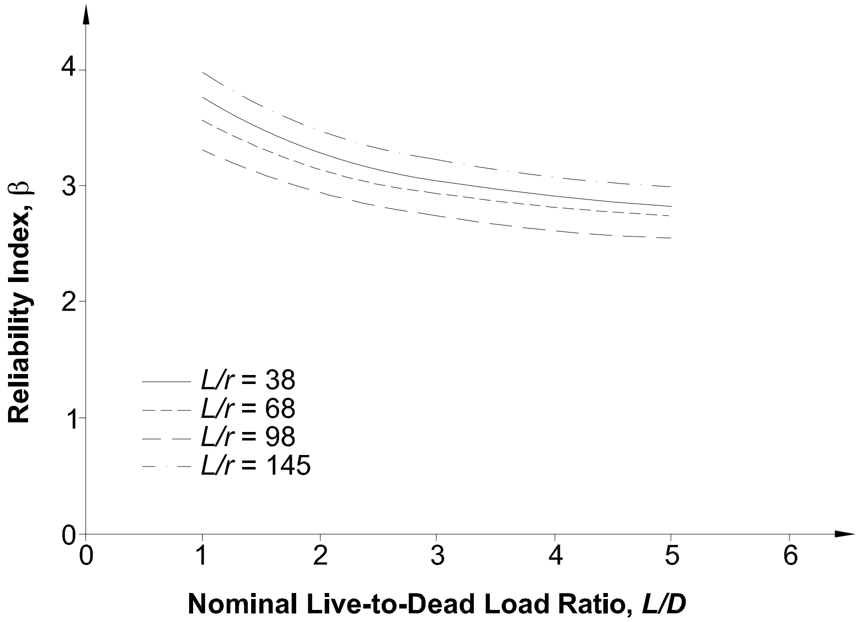


Fig. C-E1.2. Reliability of columns (ASD).

E2. SLENDERNESS LIMITATIONS AND EFFECTIVE LENGTH

The concept of a maximum limiting slenderness ratio has experienced an evolutionary change from a mandatory "... The slenderness ratio, *KL/r*, of compression members shall not exceed 200 ... " in the 1978 Specification to no restriction at all in this Specification. The 1978 *ASD* and the 1999 *LRFD Specifications* (AISC, 1978; AISC, 2000b) provide a transition from the rigid mandatory limit to no limit by the flexible provision that "... the slenderness ratio, KL/r, preferably should not exceed 200. ..." This latter restriction is actually no limit at all, so the present Specification has disposed with the provision altogether. However, the designer should keep in mind that columns with a slenderness ratio of more than 200 will have a critical stress (Equation E3-4) less than 6.3 ksi (43.5 MPa). The traditional upper limit of 200 was based on professional judgment and practical construction economics, ease of handling, and care required to minimize inadvertent damage during fabrication, transport and erection. It is not recommended to exceed this limit for compression members except for cases where special care is exercised by the fabricator and erector.

E3. COMPRESSIVE STRENGTH FOR FLEXURAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Section E3 applies to compression members with compact and noncompact sections, as defined in Section B4.

The column strength equations in Section E3 are the same as those in the previous editions of the LRFD Specification, with the exception of the cosmetic replacement of the nondimensional slenderness ratio $\lambda_c = \frac{KL}{\pi r}\sqrt{\frac{F_y}{E}}$ by the more familiar $\frac{KL}{r}$. For the convenience of those calculating the elastic buckling stress directly, without first calculating K, the limits on use of Equations E3-2 and E3-3 are also provided in terms of F_e.

Comparisons between the previous column design curves and the new one are shown in Figures C-E3.1 and C-E3.2 for the case of $F_y = 50$ ksi (345 MPa). The curves show the variation of the available column strength with the slenderness ratio for LRFD and ASD, respectively. The LRFD curves reflect the change of the resistance factor ϕ from 0.85 to 0.90, as was explained in Commentary Section E1 above. For both LRFD and ASD, the new column equations give somewhat more economy than the previous editions of the Specification.

The limit between elastic and inelastic buckling is defined to be $\frac{KL}{r} = 4.71\sqrt{\frac{E}{F_y}}$ or $F_e = 0.44F_y$. For convenience, these limits are defined in Table C-E3.1 for the common values of F_y.

One of the key parameters in the column strength equations is the elastic critical stress, F_e. Equation E3-4 presents the familiar Euler form for F_e. However, F_e can be determined by other means also, including a direct frame buckling analysis, as permitted in Chapter C, or from a torsional or *flexural-torsional* buckling analysis addressed in Section E4.

The column strength equations of Section E3 are generic equations that can be used for frame buckling and for torsional or flexural-torsional buckling (Section E4); they can also be entered with a modified slenderness ratio for single-angle members (Section E5); and they can be modified by the Q-factor for columns with slender elements (Section E7).

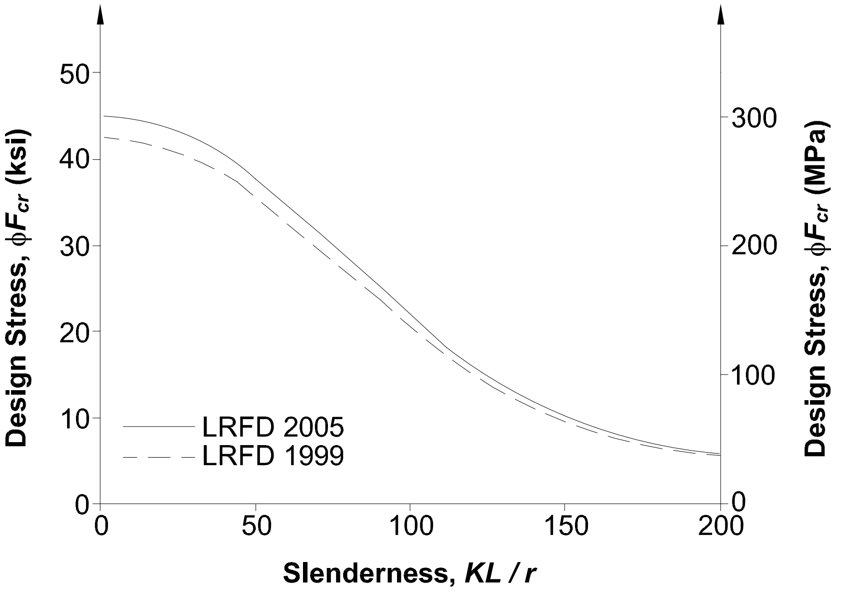


Fig. C-E3.1. LRFD column curves compared.

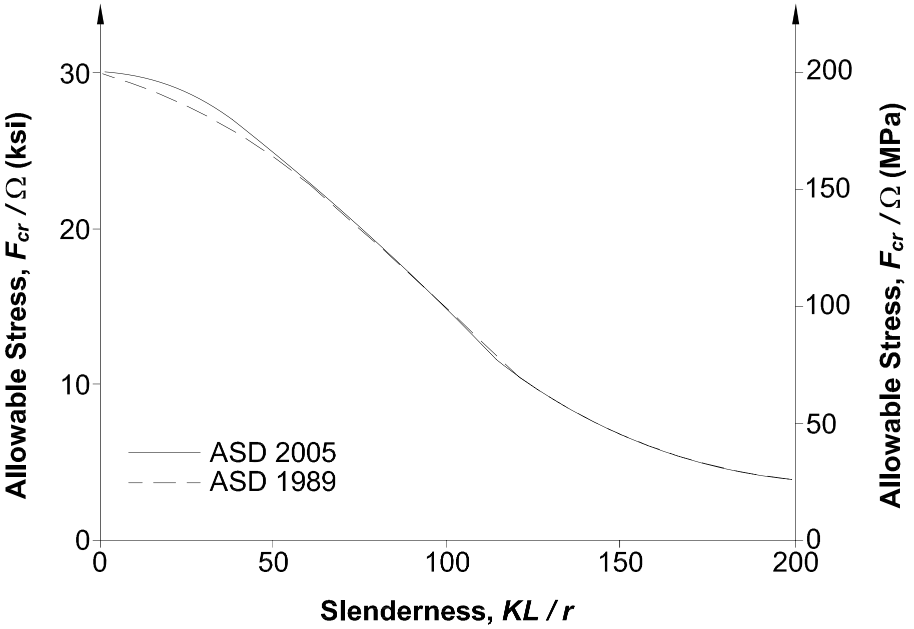


Fig. C-E3.2. ASD column curves compared.

TABLE C-E3.1
Limiting Values of KL/r and F_e

F_y ksi (MPa)	Limiting $\frac{KL}{r}$	F_e ksi (MPa)
36 (248)	134	15.8 (109)
50 (345)	113	22.0 (152)
60 (414)	104	26.4 (182)
70 (483)	96	30.8 (212)

E4. COMPRESSIVE STRENGTH FOR TORSIONAL AND FLEXURAL-TORSIONAL BUCKLING OF MEMBERS WITHOUT SLENDER ELEMENTS

Section E4 applies to singly symmetric and unsymmetric members, and certain doubly symmetric members, such as cruciform or built-up columns, with compact and noncompact sections, as defined in Section B4 for uniformly compressed elements.

The equations in Section E4 for determining the torsional and *flexural-torsional* elastic buckling loads of columns are derived in texts on structural stability [for example, Timoshenko and Gere (1961); Bleich (1952); Galambos (1968); Chen and Atsuta (1977)]. Since these equations apply only to elastic buckling, they must be modified for inelastic buckling by using the torsional and flexural-torsional critical stress, F_{cr}, in the column equations of Section E3.

Torsional buckling of symmetric shapes and flexural-torsional buckling of unsymmetrical shapes are failure modes usually not considered in the design of hot-rolled columns. They generally do not govern, or the *critical load* differs very little from the weak-axis planar buckling load. Torsional and flexural-torsional buckling modes may, however, control the strength of symmetric columns manufactured from relatively thin plate elements and unsymmetric columns and symmetric columns having torsional unbraced lengths significantly larger than the weak-axis flexural unbraced lengths. Equations for determining the critical stress for such columns are given in Section E4. Table C-E4.1 serves as a guide for selecting the appropriate equations.

The simpler method of calculating the buckling strength of double-angle and T-shaped members (Equation E4-2) uses directly the y-axis flexural strength from the column equations of Section E3 (Galambos, 1991). Tees that conform to the limits of Table C-E4.2 need not be checked for flexural-torsional buckling.

Equations E4-4 and E4-11 contain a torsional buckling effective length factor K_z. This factor may be conservatively taken as $K_z = 1.0$. For greater accuracy, $K_z = 0.5$ if both ends of the column have a connection that restrains warping, say by boxing the end over a length at least equal to the depth of the member. If one end

TABLE C-E4.1
Selection of Equations for Torsional and Flexural-Torsional Buckling

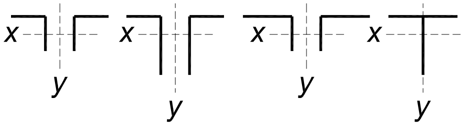
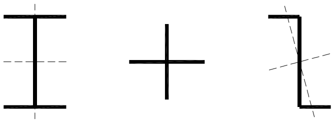
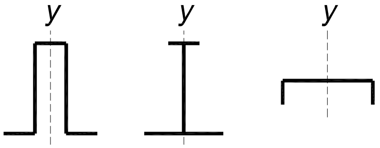
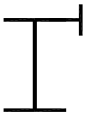
Type of Cross Section	Applicable Equations in Section E4
Double angle and T-shaped members—Case (a) in Section E4. 	E4-2 and E4-3
All doubly symmetric shapes and Z-shapes—Case (b)(i) 	E4-4
Singly symmetric members except double angles and T-shaped members—Case (b)(ii) 	E4-5
Unsymmetrical shapes—Case (b)(iii) 	E4-6

TABLE C-E4.2
Limiting Proportions for Tees

Shape	Ratio of Full Flange Width to Profile Depth	Ratio of Flange Thickness to Stem Thickness
Built-up tees	≥ 0.50	≥ 1.25
Rolled tees	≥ 0.50	≥ 1.10

of the member is restrained from warping and the other end is free to warp, then $K_z = 0.7$.

At points of bracing both lateral and/or torsional bracing shall be provided, as required in Appendix 6. Seaburg and Carter (1997) provides an overview of the fundamentals of torsional loading for structural steel members. Design examples are also included.

E5. SINGLE-ANGLE COMPRESSION MEMBERS

Section E5 addresses the design of single angles subjected to an axial compressive load effect introduced through one connected leg. The attached leg is to be fixed to a gusset plate or the projecting leg of another member by welding or by a bolted connection with at least two bolts. The equivalent slenderness expressions in this section presume significant restraint about the y-axis, which is perpendicular to the connected leg. This leads to the angle member tending to bend and buckle primarily about the x-axis. For this reason L/r_x is the slenderness parameter used. The modified slenderness ratios indirectly account for bending in the angles due to the eccentricity of loading and for the effects of end restraint from the truss chords. The values for box trusses reflect greater rotational end restraint as compared to that provided by planar trusses.

The equivalent slenderness expressions also presume a degree of rotational restraint. Equations E5-3 and E5-4 [Case (b)] assume a higher degree of x-axis rotational restraint than do Equations E5-1 and E5-2 [Case (a)]. Equations E5-3 and E5-4 are essentially equivalent to those employed for equal-leg angles as web members in latticed transmission towers in ASCE 10-97 (ASCE, 2000).

In space trusses, the web members framing in from one face typically restrain the twist of the chord at the panel points and thus provide significant x-axis restraint of the angles under consideration. It is possible that the chords of a planar truss well restrained against twist justify use of Case (b), in other words, Equations E5-3 and E5-4. Similarly, simple single-angle diagonal braces in braced frames could be considered to have enough end restraint such that Case (a), in other words, Equations E5-1 and E5-2 could be employed for their design. This procedure, however, is not intended for the evaluation of the compressive strength of x-braced single angles.

The procedure in Section E5 permits use of unequal-leg angles attached by the smaller leg provided that the equivalent slenderness is increased by an amount that is a function of the ratio of the longer to the shorter leg lengths, and has an upper limit on L/r_z.

If the single-angle compression members cannot be evaluated using the procedures in this section, use the provisions of Section H2. In evaluating P_n, the effective length due to end restraint should be considered. With values of effective length factors about the geometric axes, one can use the procedure in Lutz (1992) to compute an effective radius of gyration for the column. To obtain results that are not too conservative, one must also consider that end restraint reduces the eccentricity of the axial load of single-angle struts and thus the value of f_b used in the flexural term(s) in Equation H2-1.

E6. BUILT-UP MEMBERS

Section E6 addresses the strength and dimensional requirements of built-up members composed of two or more shapes interconnected by stitch bolts or welds.

1. Compressive Strength

The longitudinal spacing of connectors connecting components of built-up compression members must be such that the slenderness ratio L/r of individual shapes does not exceed three-fourths of the slenderness ratio of the entire member. However, this requirement does not necessarily ensure that the effective slenderness ratio of the built-up member is equal to that of a built-up member acting as a single unit. Section E6.1 gives equations for modified slenderness ratios that are based on research and take into account the effect of shear deformation in the connectors (Zandonini, 1985). Equation E6-1 for snug-tight intermediate connectors is empirically based on test results. Equation E6-2 is derived from theory and verified by test data. In both cases the end connection must be welded or fully tensioned bolted (Aslani and Goel, 1991). The connectors must be designed to resist the shear forces that develop in the buckled member. The shear stresses are highest where the slope of the buckled member is the steepest (Bleich, 1952). Fastener spacing less than the maximum required for strength may be needed to ensure a close fit over the entire faying surface of components in continuous contact. Special requirements for weathering steel members exposed to atmospheric corrosion are given in Brockenbrough (1983).

2. Dimensional Requirements

Section E6.2 provides requirements for dimensioning built-up members that cannot be stated in terms of calculated stress but are based upon judgment and experience.

E7. MEMBERS WITH SLENDER ELEMENTS

The structural engineer designing with hot-rolled plates and shapes will seldom find an occasion to turn to Section E7 of the Specification. Among rolled shapes the most frequently encountered cases requiring the application of this section are columns containing angles with thin legs and tee-shaped columns having slender stems. Special attention to the determination of Q must be given when columns are made by welding or bolting thin plates together.

The provisions of Section E7 address the modifications to be made when one or more plate elements in the column cross sections are slender. A plate element is considered to be slender if its width-thickness ratio exceeds the limiting value λ_r defined in Table B4.1. As long as the plate element is not slender, it can support the full yield stress without local buckling. When the cross section contains slender elements, the slenderness reduction factor Q defines the ratio of the stress at local buckling to the yield stress, F_y. The yield stress, F_y, is replaced by the value QF_y in the column equations of Section E3. These equations are repeated as Equations E7-2 and E7-3. This approach to dealing with columns with slender elements has been used since the 1969 Specification (AISC, 1969), emulating the 1969 AISI Specification (AISI, 1969). Prior to 1969, the AISC practice was to remove the width of the plate that exceeded the limit λ_r and check the remaining cross section for conformance with the allowable stress, which proved inefficient and uneconomical. The equations in Section E7 are almost identical to the original equations, with one notable exception that will be discussed subsequently.

This Specification makes a distinction between columns having unstiffened and stiffened elements. Two separate philosophies are used: Unstiffened elements are considered to have attained their limit state when they reach the theoretical local buckling stress. Stiffened elements, on the other hand, make use of the post-buckling strength inherent in a plate that is supported on both of its longitudinal edges, such as in HSS columns. The effective width concept is used to obtain the added post-buckling strength. This dual philosophy reflects 1969 practice in the design of cold-formed columns. Subsequent editions of the AISI Specifications, in particular, the *North American Specification for the Design of Cold-Formed Steel Structural Members* (AISI, 2001), hereafter referred to as the AISI *North American Specification* adopted the effective width concept for both stiffened and unstiffened columns. Following editions of the AISC Specification (including this Specification) did not follow the example set by AISI for unstiffened plates because the advantages of the post-buckling strength do not become available unless the plate elements are very slender. Such dimensions are common for cold-formed columns, but are rarely encountered in structures made from hot-rolled plates.

1. Slender Unstiffened Elements, Q_s

Equations for the slender element reduction factor, Q_s, are given in Section E7.1 for outstanding elements in rolled shapes (Case a), built-up shapes (Case b), single angles (Case c), and stems of tees (Case d). The underlying scheme for these provisions is illustrated in Figure C-E7.1. The curves show the relationship between the Q-factor and a non-dimensional slenderness ratio $\frac{b}{t}\sqrt{\frac{F_y}{E}\frac{12(1-\nu^2)}{\pi^2 k}}$.

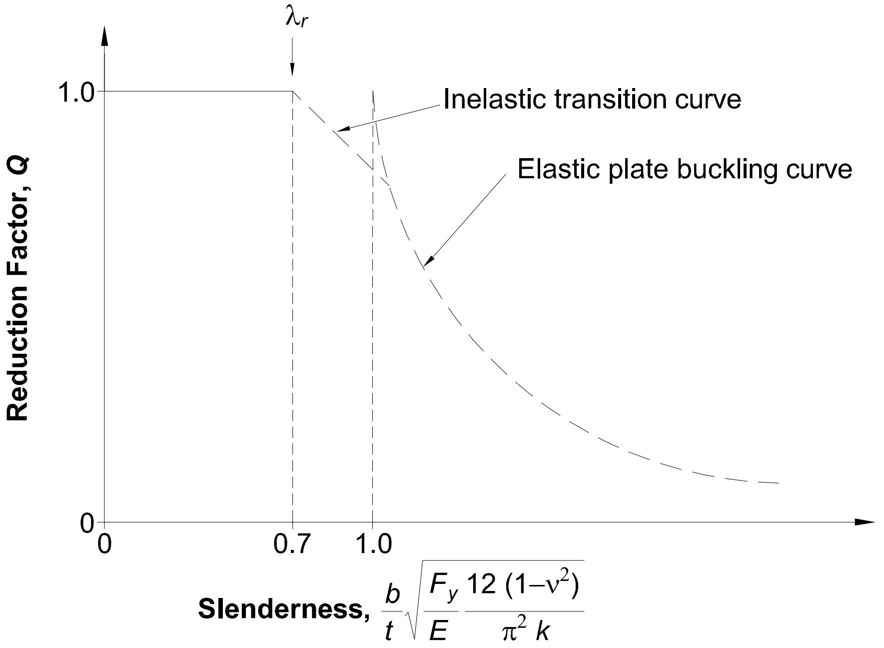


Fig. C-E7.1. Definition of Q_s for unstiffened slender elements.

The width b and thickness t are defined for the applicable cross sections in Section B4; $\nu = 0.3$ (Poisson's ratio), and k is the plate buckling coefficient characteristic of the type of plate edge-restraint. For single angles, $k = 0.425$ (no restraint is assumed from the other leg), and for outstanding flange elements and stems of tees, k equals approximately 0.7, reflecting an estimated restraint from the part of the cross section to which the plate is attached on one of its edges, the other edge being free.

The curve relating Q to the plate slenderness ratio has three components: (i) a part where $Q = 1$ when the slenderness factor is less than or equal to 0.7 (the plate can be stressed up to its yield stress), (ii) the elastic plate buckling portion when buckling is governed by $F_{cr} = \dfrac{\pi^2 Ek}{12(1-\nu^2)\left(\frac{b}{t}\right)^2}$, and (iii) a transition range that empirically accounts for the effect of early yielding due to *residual stresses* in the shape. Generally this transition range is taken as a straight line. The development of the provisions for unstiffened elements is due to the research of Winter and his co-workers, and a full listing of references is provided in the Commentary to the AISI *North American Specification* (AISI, 2001). The slenderness provisions are illustrated for the example of slender flanges of rolled shapes in Figure C-E7.2.

The equations for the unstiffened projecting flanges, angles and plates in built-up cross sections (Equations E7-7 through E7-9) have a history that starts with the research reported in Johnson (1985). It was noted in tests of beams with slender flanges and slender webs that there was an interaction between the buckling of the flanges and the distortions in the web causing an unconservative prediction of strength. A modification based on the equations recommended in Johnson (1985) appeared first in the 1989 *ASD Specification* (AISC, 1989).

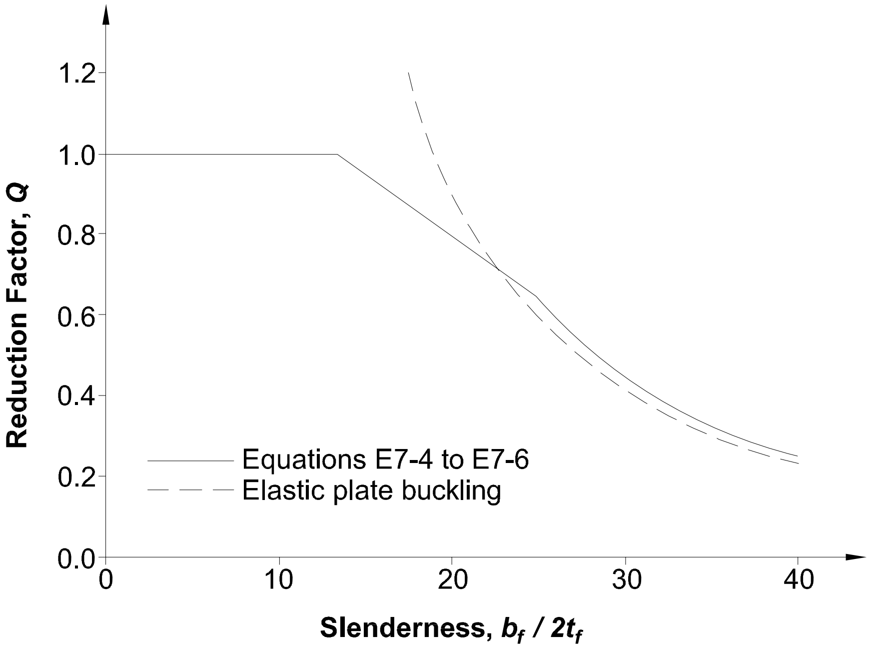


Fig. C-E7.2. Q for rolled wide-flange columns of $F_y = 50$ *ksi (345 MPa).*

Modifications to simplify the original equations were introduced in the 1993 *LRFD Specification* (AISC, 1993), and these equations have remained unchanged in the present Specification. The influence of web slenderness is accounted for by the introduction of the factor

$$k_c = \frac{4}{\sqrt{\dfrac{h}{t_w}}} \tag{C-E7-1}$$

into the equations for λ_r and Q, where k_c shall not be taken less than 0.35 nor greater than 0.76 for calculation purposes.

2. Slender Stiffened Elements, Q_a

While for slender unstiffened elements the Specification for local buckling is based on the limit state of the onset of plate buckling, an improved approach based on the effective width concept is used for the compressive strength of stiffened elements in columns. This method was first proposed in von Kármán, Sechler, and Donnell (1932). This was later modified in Winter (1947) to provide a transition between very slender elements and stockier elements shown by tests to be fully effective. As modified in Winter (1947) for the AISI *North American Specification* (AISI, 2001), the ratio of effective width to actual width increases as the level of compressive stress applied to a stiffened element in a member is decreased, and takes the form

$$\frac{b_e}{t} = 1.9\sqrt{\frac{E}{f}}\left[1 - \frac{C}{(b/t)}\sqrt{\frac{E}{f}}\right] \tag{C-E7-2}$$

where f is taken as F_{cr} of the column based on $Q = 1.0$, and C is a constant based on test results (Winter, 1947).

The basis for cold-formed steel columns in the AISI *North American Specification* editions since the 1970s is $C = 0.415$. The original AISI coefficient 1.9 in Equation C-E7-2 is changed to 1.92 in the Specification to reflect the fact that the modulus of elasticity E is taken as 29,500 ksi (203 400 MPa) for cold-formed steel, and 29,000 ksi (200 000 MPa) for hot-rolled steel.

For the case of square and rectangular box-sections of uniform thickness, where the sides provide negligible rotational restraint to one another, the value of $C = 0.38$ in Equation E7-18 is higher than the value of $C = 0.34$ in Equation E7-17. Equation E7-17 applies to the general case of stiffened plates in uniform compression where there is substantial restraint from the adjacent flange or web elements. The coefficients $C = 0.38$ and $C = 0.34$ are smaller than the corresponding value of $C = 0.415$ in the AISI *North American Specification* (AISI, 2001), reflecting the fact that hot-rolled steel sections have stiffer connections between plates due to welding or fillets in rolled shapes than do cold-formed shapes.

The classical theory of longitudinally compressed cylinders overestimates the actual buckling strength, often by 200 percent or more. Inevitable imperfections of shape and the eccentricity of the load are responsible for the reduction in actual strength below the theoretical strength. The limits in Section E7.2(c) are based upon test evidence (Sherman, 1976), rather than theoretical calculations, that local buckling will not occur if

$$\frac{D}{t} \leq \frac{0.11E}{F_y}$$

When D/t exceeds this value but is less than

$$\frac{D}{t} \leq \frac{0.45E}{F_y}$$

Equation E7-19 provides a reduction in the local buckling reduction factor Q. This Specification does not recommend the use of round HSS or pipe columns with

$$\frac{D}{t} > \frac{0.45E}{F_y}$$

CHAPTER F

DESIGN OF MEMBERS FOR FLEXURE

F1. GENERAL PROVISIONS

Chapter F applies to members subject to simple bending about one principal axis of the cross section. Section F2 gives the provisions for the flexural strength of doubly symmetric compact I-shaped and channel members subject to bending about their major axis. For most designers, the provisions in this section will be sufficient to perform their everyday designs. The remaining sections of Chapter F address less frequently occurring cases encountered by structural engineers. Since there are many such cases, many equations and many pages in the Specification, the table in User Note F1.1 is provided as a map for navigating through the cases considered in Chapter F. The coverage of the chapter is extensive and there are many equations that appear formidable; however, it is stressed again that for most designs, the engineer need seldom go beyond Section F2.

For all sections covered in Chapter F, the highest possible nominal flexural strength is the plastic moment, $M_n = M_p$. Being able to use this value in design represents the optimum use of the steel. In order to attain M_p the beam cross section must be compact and the member must be laterally braced. Compactness depends on the flange and web plate width-to-thickness ratios, as defined in Section B4. When these conditions are not met, the available nominal flexural strength diminishes. All sections in Chapter F treat this reduction in the same way. For laterally braced beams, the plastic moment region extends over the range of plate width-thickness ratios λ terminating at λ_p. This is the compact condition. Beyond these limits the nominal moment reduces linearly until λ reaches λ_r. This is the range where the section is noncompact. Beyond λ_r the section is a slender-element section.

These three ranges are illustrated in Figure C-F1.1 for the case of rolled wide-flange members for the limit state of flange local buckling. The curve in Figure C-F1.1 shows the relationship between the flange width-thickness ratio $b_f/2t_f$ and the nominal flexural strength, M_n.

The basic relationship between the nominal flexural strength, M_n, and the unbraced length, L_b, for the limit state of lateral-torsional buckling is shown in Figure C-F1.2 for a compact section [W27×84 (W690×125), $F_y = 50$ ksi (345 MPa)] subjected to uniform bending, $C_b = 1.0$.

There are four principal zones defined on the basic curve by the lengths L_{pd}, L_p, and L_r. Equation F2-5 defines the maximum unbraced length L_p to reach M_p

with uniform moment. Elastic lateral-torsional buckling will occur when the unbraced length is greater than L_r given by Equation F2-6. Equation F2-2 defines the range of inelastic lateral-torsional buckling as a straight line between the defined limits M_p at L_p and $0.7F_yS_x$ at L_r. Buckling strength in the elastic region is given by Equations F2-3 and F2-4 for I-shaped members. The length

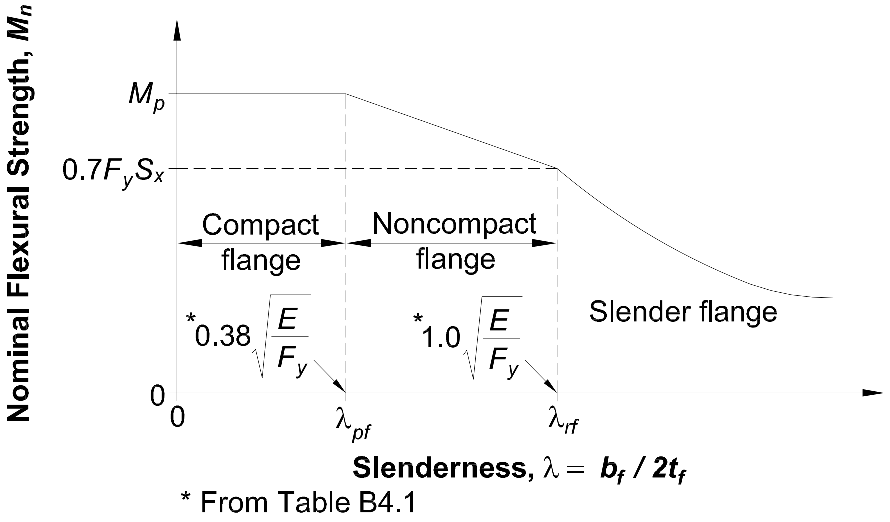


Fig. C-F1.1. Nominal flexural strength as a function of the flange width-thickness ratio of rolled I-shapes.

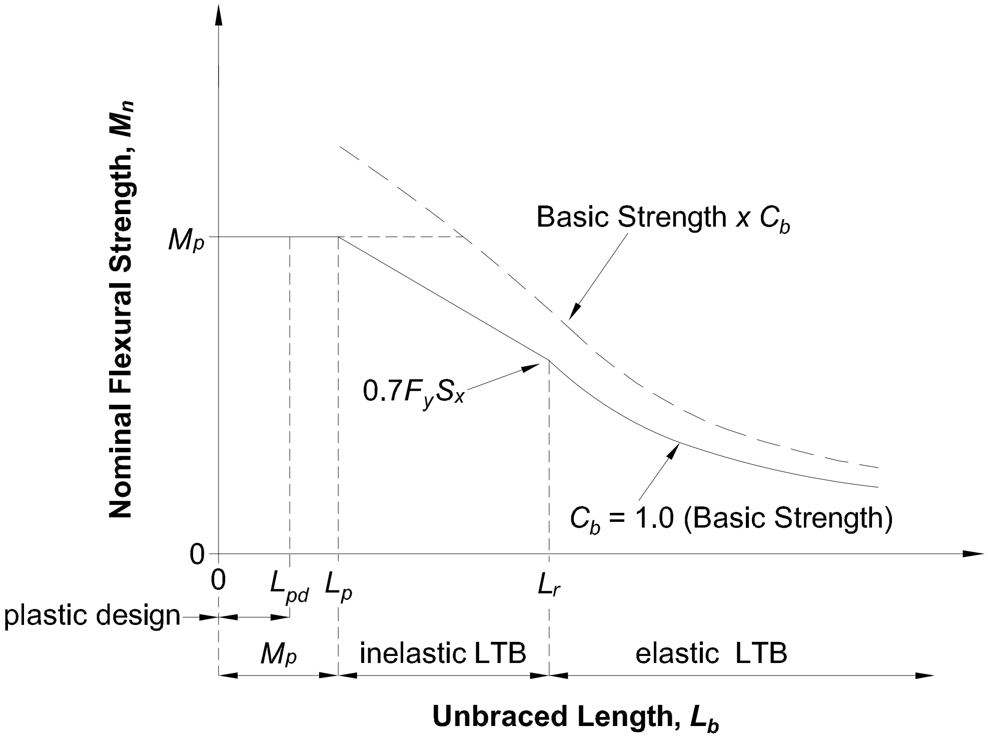


Fig. C-F1.2. Nominal flexural strength as a function of unbraced length and moment gradient.

L_{pd} is defined in Appendix 1 as the limiting unbraced length needed for plastic design.

For moment diagrams along the member other than uniform moment, the lateral buckling strength is obtained by multiplying the basic strength in the elastic and inelastic region by C_b as shown in Figure C-F1.2. However, in no case can the maximum moment capacity exceed the plastic moment M_p. Note that L_p given by Equation F2-5 is merely a definition that has physical meaning only when $C_b = 1.0$. For C_b greater than 1.0, members with larger unbraced lengths can reach M_p, as shown by the curve for $C_b > 1.0$ in Figure C-F1.2. This length is calculated by setting Equation F2-2 equal to M_p and solving for L_b using the actual value of C_b.

The equation

$$C_b = 1.75 + 1.05\left(\frac{M_1}{M_2}\right) + 0.3\left(\frac{M_1}{M_2}\right)^2 \qquad \text{(C-F1-1)}$$

has been used since 1961 in AISC Specifications to adjust the lateral-torsional buckling equations for variations in the moment diagram within the unbraced length. However, this equation is only applicable to moment diagrams that consist of straight lines between braced points—a condition that is rare in beam design. The equation provides a lower bound to the solutions developed in Salvadori (1956). Equation C-F1-1 can be easily misinterpreted and misapplied to moment diagrams that are not linear within the unbraced segment. Kirby and Nethercot (1979) present an equation that applies to various shapes of moment diagrams within the unbraced segment. Their original equation has been slightly adjusted to give Equation C-F1-2 (Equation F1-1 in the body of the Specification):

$$C_b = \frac{12.5M_{max}}{2.5M_{max} + 3M_A + 4M_B + 3M_C} \qquad \text{(C-F1-2)}$$

This equation gives a more accurate solution for a fixed-end beam, and gives approximately the same answers as Equation C-F1-1 for moment diagrams with straight lines between points of bracing. C_b computed by Equation C-F1-2 for moment diagrams with other shapes show good comparison with the more precise but also more complex equations (Galambos, 1998). The absolute values of the three quarter-point moments and the maximum moment regardless of its location are used in Equation C-F1-2. The maximum moment in the unbraced segment is always used for comparison with the nominal moment M_n. The length between braces, not the distance to inflection points is used. It is still satisfactory to use C_b from Equation C-F1-1 for straight-line moment diagrams within the unbraced length.

The equations for the limit state of lateral-torsional buckling in Chapter F assume that the loads are applied along the beam centroidal axis. C_b may be conservatively taken equal to 1.0, with the exception of some cases involving unbraced cantilevers or members with no bracing within the span and with significant loading applied to the top flange. If the load is placed on the top flange and the flange is not braced, there is a tipping effect that reduces the critical moment; conversely, if the load is suspended from an unbraced bottom flange, there is a stabilizing effect that increases the critical moment (Galambos, 1998). For unbraced top flange loading on compact I-shaped members, the reduced critical moment may be conservatively approximated by setting the square root expression in Equation F2-4 equal to unity.

An effective length factor of unity is implied in the critical moment equations to represent the worst-case simply supported unbraced segment. Consideration of any end restraint due to adjacent nonbuckled segments on the critical segment can increase its strength. The effects of beam continuity on lateral-torsional buckling have been studied, and a simple conservative design method, based on the analogy to end-restrained nonsway columns with an effective length less than unity, has been proposed (Galambos, 1998).

F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

Section F2 applies to members with compact I-shaped or channel cross sections subject to bending about their major axis; hence the only limit state to consider is lateral-torsional buckling. Almost all rolled wide-flange shapes listed in the AISC *Manual of Steel Construction* are eligible to be designed by the provisions of this section, as indicated in the User Note in the Specification.

The equations in Section F2 are identical to the corresponding equations in Section F1 of the 1999 *LRFD Specification* (AISC, 2000b), although they are presented in different form. The following table gives the list of equivalent equations:

TABLE C-F2.1
Comparison of Equations for Nominal Flexural Strength

1999 AISC/LRFD Specification Equations	Current Specification Equations
F1-1	F2-1
F1-2	F2-2
F1-13	F2-3 and F2-4

The only difference between the two specifications is that the stress at the interface between inelastic and elastic buckling has been changed from $F_y - F_r$ in the 1999 edition to $0.7F_y$ herein. In the previous Specification the residual stress, F_r, for rolled and welded shapes was different, namely 10 ksi (69 MPa) and 16.5 ksi (114 MPa), respectively, while in this Specification the *residual stress* was taken as $0.3F_y$ so that the value of $F_y - F_r = 0.7F_y$ was adopted. This change was made in the interest of simplicity with negligible effect on economy.

The elastic lateral-torsional buckling stress, F_{cr}, of Equation F2-4:

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \sqrt{1 + 0.078 \frac{Jc}{S_x h_o} \left(\frac{L_b}{r_{ts}}\right)^2} \qquad \text{(C-F2-1)}$$

is identical to Equation F1-13 in the 1999 *LRFD Specification* (AISC, 2000b):

$$F_{cr} = \frac{M_{cr}}{S_x} = \frac{C_b \pi}{L_b S_x} \sqrt{EI_y GJ + \left(\frac{\pi E}{L_b}\right)^2 I_y C_w} \qquad \text{(C-F2-2)}$$

if $c = 1$ (see Section F2 for definition) and

$$r_{ts}{}^2 = \frac{\sqrt{I_y C_w}}{S_x}; \quad h_o = d - t_f; \quad \text{and } \frac{2G}{\pi^2 E} = 0.0779$$

Equation F2-5 is the same as F1-4 in the 1999 *LRFD Specification* (AISC, 2000b), and F2-6 corresponds to F1-6. It is obtained by setting $F_{cr} = 0.7F_y$ in Equation F2-4 and solving for L_b. The term r_{ts} can conservatively be calculated as the radius of gyration of the compression flange plus one-sixth of the web.

These provisions have been simplified when compared to the previous ASD provisions based on a more informed understanding of beam limit states behavior. The maximum allowable stress obtained in these provisions may be slightly higher than the previous limit of $0.66F_y$, since the true plastic strength of the member is reflected by use of the plastic section modulus in Equation F2-1. The Section F2 provisions for unbraced length are satisfied through the use of two equations, one for inelastic lateral-torsional buckling (Equation F2-2), and one for elastic lateral-torsional buckling (Equation F2-3). Previous ASD provisions placed an arbitrary stress limit of $0.6F_y$ when a beam was not fully braced and required that three equations be checked with the selection of the largest stress to determine the strength of a laterally unbraced beam. With the current provisions, once the unbraced length is determined, the member strength can be obtained directly from these equations.

F3. DOUBLY SYMMETRIC I-SHAPED MEMBERS WITH COMPACT WEBS AND NONCOMPACT OR SLENDER FLANGES BENT ABOUT THEIR MAJOR AXIS

Section F3 is a supplement to Section F2 for the case where the flange of the section is noncompact or slender (see Figure C-F1.1, linear variation of M_n

between λ_{pf} and λ_{rf}). As pointed out in the user note of Section F2, very few rolled wide-flange shapes are subject to this criterion.

F4. OTHER I-SHAPED MEMBERS WITH COMPACT OR NONCOMPACT WEBS BENT ABOUT THEIR MAJOR AXIS

Section F4 has no direct counterpart in previous AISC Specifications except for the lateral buckling provisions for singly symmetric sections in Table A-F1.1 in the 1999 *LRFD Specification* (AISC, 2000b). These provisions are not carried over to the present Specification. The provisions of Section F4 are applicable to doubly symmetric wide-flange beams with slender flanges and to singly symmetric wide-flange members with compact, noncompact, and slender flanges, and noncompact webs (see the Table in User Note F1.1). This part of Chapter F essentially deals with welded I-shaped beams where the webs are not slender. The following section, F5, considers welded I-shapes with slender webs. The contents of Section F4 are based on White (2004).

Three limit states are considered: (a) lateral-torsional buckling (LTB); (b) flange local buckling (FLB); and (c) tension flange yielding (TFY). The effect of inelastic buckling of the web is taken care of indirectly by multiplying the moment causing yielding in the compression flange by a factor R_{pc} and the moment causing yielding in the tension flange by a factor R_{pt}. These two factors can vary from unity to as high as 1.6. Conservatively, they can be assumed to equal 1.0. The following steps are provided as a guide to the determination of R_{pc} and R_{pt}.

Step 1. Calculate h_p and h_c: See Figure C-F4.1.

Step 2. Determine web slenderness and yield moments in compression and tension:

$$\left\{\begin{array}{l} \lambda = \dfrac{h_c}{t_w} \\ S_{xc} = \dfrac{I_x}{y}; \quad S_{xt} = \dfrac{I_x}{d-y} \\ M_{yc} = F_y S_{xc}; \quad M_{yt} = F_y S_{xt} \end{array}\right\} \tag{C-F4-1}$$

Step 3. Determine λ_{pw} and λ_{rw}

$$\left\{\begin{array}{l} \lambda_{pw} = \dfrac{\dfrac{h_c}{h_p}\sqrt{\dfrac{E}{F_y}}}{\left[\dfrac{0.54M_p}{M_y} - 0.09\right]^2} \le 3.76\sqrt{\dfrac{E}{F_y}} \\ \lambda_{rw} = 5.70\sqrt{\dfrac{E}{F_y}} \end{array}\right\} \tag{C-F4-2}$$

If $\lambda > \lambda_{rw}$ then the web is slender and the design is governed by Section F5.

Step 4. Calculate R_{pc} and R_{pt} by Equations F4-9a or F4-9b and F4-15a or F4-15b, respectively.

The basic maximum nominal moment is $R_{pc}M_{yc} = R_{pc}F_yS_{xc}$ if the flange is in compression, and $R_{pt}M_{yt} = R_{pt}F_yS_{xt}$ if it is in tension. Thereafter, the provisions are the same as for doubly symmetric members in Sections F2 and F3. For the limit state of lateral-torsional buckling, I-shaped members with cross sections that have unequal flanges are treated as if they were doubly symmetric I-shapes. That is, Equations F2-4 and F2-6 are the same as Equations F4-5 and F4-8, except the former use S_x and the latter use S_{xc}, the elastic section moduli of the entire section and of the compression side, respectively. This is a simplification that tends to be somewhat conservative if the compression flange is smaller than the tension flange, and it is somewhat unconservative when the reverse is true. It is also required to check for tension flange yielding if the tension flange is smaller than the compression flange (Section F4.3).

For a more accurate solution, especially when the loads are not applied at the centroid of the member, the designer is directed to Chapter 5 of the SSRC Guide (Galambos, 1998; Galambos, 2001; White and Jung, 2003). White gives the following alternative equations in lieu of Equations F4-5 and F4-8:

$$M_n = C_b \frac{\pi^2 EI_y}{L_b^2}\left\{\frac{\beta_x}{2} + \sqrt{\left(\frac{\beta_x}{2}\right)^2 + \frac{C_w}{I_y}\left[1 + 0.0390\frac{J}{C_w}L_b^2\right]}\right\} \qquad \text{(C-F4-3)}$$

$$L_r = \frac{1.38E\sqrt{I_yJ}}{S_{xc}F_{yr}}\sqrt{\frac{2.6\beta_x F_{yr}S_{xc}}{EJ} + 1 + \sqrt{\left[\frac{2.6\beta_x F_{yr}S_{xc}}{EJ} + 1\right]^2 + \frac{27.0C_w}{I_y}\left(\frac{F_{yr}S_{xc}}{EJ}\right)^2}} \qquad \text{(C-F4-4)}$$

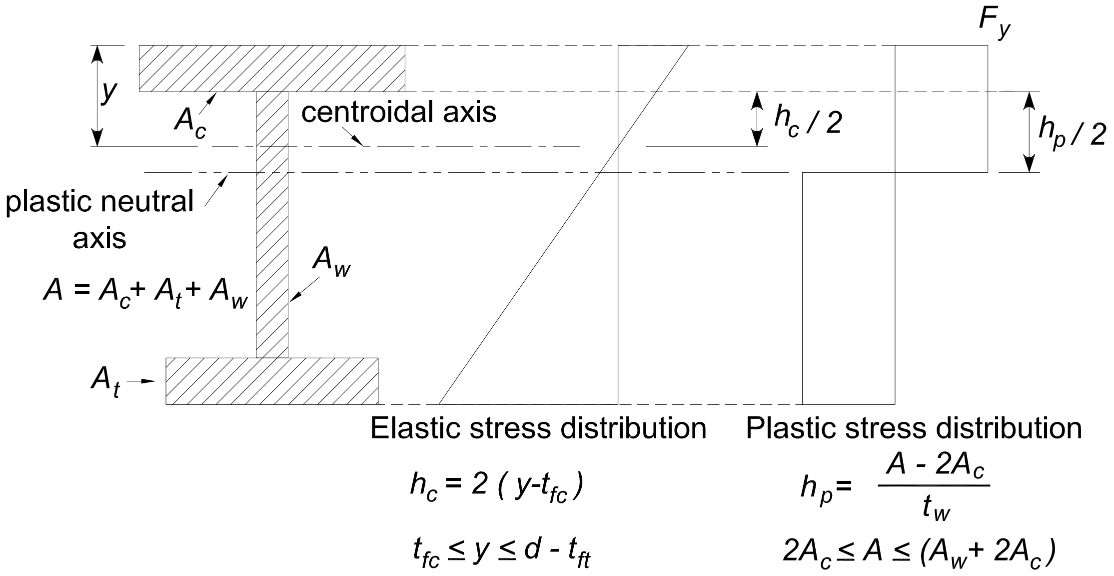


Fig. C-F4.1. Elastic and plastic stress distributions.

where the coefficient of monosymmetry, $\beta_x = 0.9h\alpha\left(\dfrac{I_{yc}}{I_{yt}} - 1\right)$,

the warping constant, $C_w = h^2 I_{yc}\alpha$, and $\alpha = \dfrac{1}{\dfrac{I_{yc}}{I_{yt}} + 1}$.

F5. DOUBLY SYMMETRIC AND SINGLY SYMMETRIC I-SHAPED MEMBERS WITH SLENDER WEBS BENT ABOUT THEIR MAJOR AXIS

This section applies for doubly and singly symmetric I-shaped welded plate girders with a slender web, that is, $\dfrac{h_c}{t_w} > \lambda_r = 5.70\sqrt{\dfrac{E}{F_y}}$. The applicable limit states are lateral-torsional buckling, compression flange local buckling and tension flange local yielding. The provisions in this section have changed little since 1963. They are similar to the provisions in Section A-G2 in the 1999 *LRFD Specification* (AISC, 2000b), and similar to the provisions in Section G2 in the 1989 *ASD Specification* (AISC, 1989). The provisions for plate girders are based on research reported in Basler and Thurlimann (1963).

There is no seamless transition between the equations in Section F4 and F5. Thus the bending strength of a girder with $F_y = 50$ ksi (345 MPa) and a web slenderness $h/t_w = 137$ is not close to that of a girder with $h/t_w = 138$. These two slenderness ratios are on either side of the limiting ratio. This gap is caused by the discontinuity between the lateral-torsional buckling resistances predicted by Section F4 and those predicted by Section F5 due to the implicit use of $J = 0$ in Section F5. However, for typical noncompact web section members close to the noncompact web limit, the influence of J on the lateral-torsional buckling resistance is relatively small (for example, the calculated L_r values including J versus using $J = 0$ typically differ by less than 10 percent). The implicit use of $J = 0$ in Section F5 is intended to account for the influence of web distortional flexibility on the lateral-torsional buckling resistance for slender-web I-section members.

F6. I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MINOR AXIS

I-shaped members and channels bent about their minor axis do not experience lateral-torsional buckling or web buckling. The only limit states to consider are yielding and flange local buckling. The user note informs the designer of the few rolled shapes that need to be checked for flange local buckling.

F7. SQUARE AND RECTANGULAR HSS AND BOX-SHAPED MEMBERS

The provisions for the nominal flexural strength of HSS include the limit states of yielding and local buckling. Square and rectangular HSS bent about the minor axis are not subject to lateral-torsional buckling.

Because of the high torsional resistance of the closed cross-section, the critical unbraced lengths L_p and L_r that correspond to the development of the plastic moment and the yield moment, respectively, are very large. For example, as shown in Figure C-F7.1, an HSS 20 × 4 × $^5/_{16}$ (HSS 508 × 101.6 × 7.9), which has one of the largest depth-width ratios among standard HSS, has L_p of 6.7 ft (2.0 m) and L_r of 137 ft (42 m) as determined in accordance with the 1993 *LRFD Specification* (AISC, 1993). An extreme deflection limit might correspond to a length-to-depth ratio of 24 or a length of 40 ft (12 m) for this member. Using the specified linear reduction between the plastic moment and the yield moment for lateral-torsional buckling, the plastic moment is reduced by only 7 percent for the 40-ft (12 m) length. In most practical designs where the moment gradient C_b is larger than unity, the reduction will be nonexistent or insignificant.

The provisions for local buckling of noncompact rectangular HSS are also the same as those in the previous sections of this chapter: $M_n = M_p$ for $b/t \leq \lambda_p$, and a linear transition from M_p to $F_y S_x$ when $\lambda_p < b/t \leq \lambda_r$. The equation for the effective width of the compression flange when b/t exceeds λ_r is the same as that used for rectangular HSS in axial compression except that the stress is taken as the yield stress. This implies that the stress in the corners of the compression flange is at yield when the ultimate post-buckling strength of the flange is reached. When using the effective width, the nominal flexural strength is determined from the effective section modulus to the compression flange using the distance from the shifted neutral axis. A slightly conservative estimate of the nominal flexural strength can be obtained by using the effective width for both the compression and tension flange, thereby maintaining the symmetry of the cross section and simplifying the calculations.

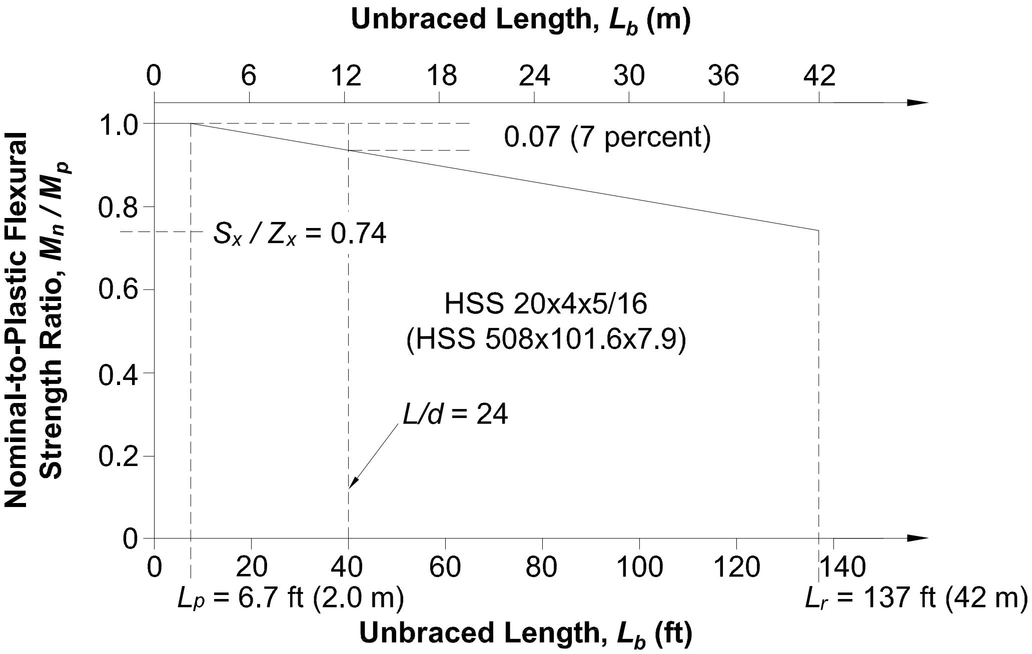


Fig. C-F7.1. Lateral-torsional buckling of rectangular HSS.

F8. ROUND HSS

Round HSS are not subject to lateral-torsional buckling. The failure modes and post-buckling behavior of round HSS can be grouped into three categories (Sherman, 1992; Galambos, 1998):

(a) For low values of D/t, a long plastic plateau occurs in the moment-rotation curve. The cross section gradually ovalizes, local wave buckles eventually form, and the moment resistance subsequently decays slowly. Flexural strength may exceed the theoretical plastic moment due to *strain hardening*.

(b) For intermediate values of D/t, the plastic moment is nearly achieved but a single local buckle develops and the flexural strength decays slowly with little or no plastic plateau region.

(c) For high values of D/t HSS, multiple buckles form suddenly with very little ovalization and the flexural strength drops quickly.

The flexural strength provisions for round HSS reflect these three regions of behavior and are based upon five experimental programs involving hot-formed seamless pipe, electric-resistance-welded pipe and fabricated tubing (Galambos, 1998).

F9. TEES AND DOUBLE ANGLES LOADED IN THE PLANE OF SYMMETRY

The lateral-torsional buckling (LTB) strength of singly symmetric tee beams is given by a fairly complex formula (Galambos, 1998). Equation F9-4 is a simplified formulation based on Kitipornchai and Trahair (1980). See also Ellifritt, Wine, Sputo, and Samuel (1992).

The C_b factor used for I-shaped beams is unconservative for tee beams with the stem in compression. For such cases $C_b = 1.0$ is appropriate. When beams are bent in reverse curvature, the portion with the stem in compression may control the LTB resistance even though the moments may be small relative to other portions of the unbraced length with $C_b \approx 1.0$. This is because the LTB strength of a tee with the stem in compression may be only about one-fourth of the strength for the stem in tension. Since the buckling strength is sensitive to the moment diagram, C_b has been conservatively taken as 1.0. In cases where the stem is in tension, connection details should be designed to minimize any end restraining moments that might cause the stem to be in compression.

No limiting stem width-thickness ratio, λ_r, is provided in this section to account for the local buckling of the stem when it is in compression. The reason for this omission is that the lateral-torsional buckling equations (Equations F9-4 and F9-5) also give the local buckling strength as L_b approaches zero. This is not immediately evident, because when $L_b = 0$ is substituted into these equations one obtains, after some algebraic manipulations, $M_{cr} = 0/0$, which is a mathematically indeterminate expression. From elementary calculus such a problem is solved by differentiating the numerator and the denominator as often as needed to arrive at an explicit expression using L'Hospital's rule. If this operation is performed twice, one can obtain the following equation for the critical moment of combined lateral-torsional and local buckling:

$$M_{cr,L_b=0} = \frac{\pi EJ\sqrt{\dfrac{G}{E}}}{4.6d} = 0.424\frac{EJ}{d} \tag{C-F9-1}$$

The relationship between the unbraced length and the critical moment for a WT18×67.5 (WT460×100.5) [$F_y = 50$ ksi (345 MPa)] tee beam, with the stem in flexural compression, is shown in Figure C-F9.1.

Although flexure about the y-axis of tees and double angles does not occur frequently, guidance is given here to address this condition. The yield limit state and the local buckling limit state of the flange can be checked by using Equations F6-1 through F6-4. Lateral-torsional buckling can conservatively be calculated by assuming the flange acts alone as a rectangular beam, using Equations F11-2 through F11-4. Alternately an elastic critical moment given as

$$M_e = \frac{\pi}{L_b}\sqrt{EI_x GJ} \tag{C-F9-2}$$

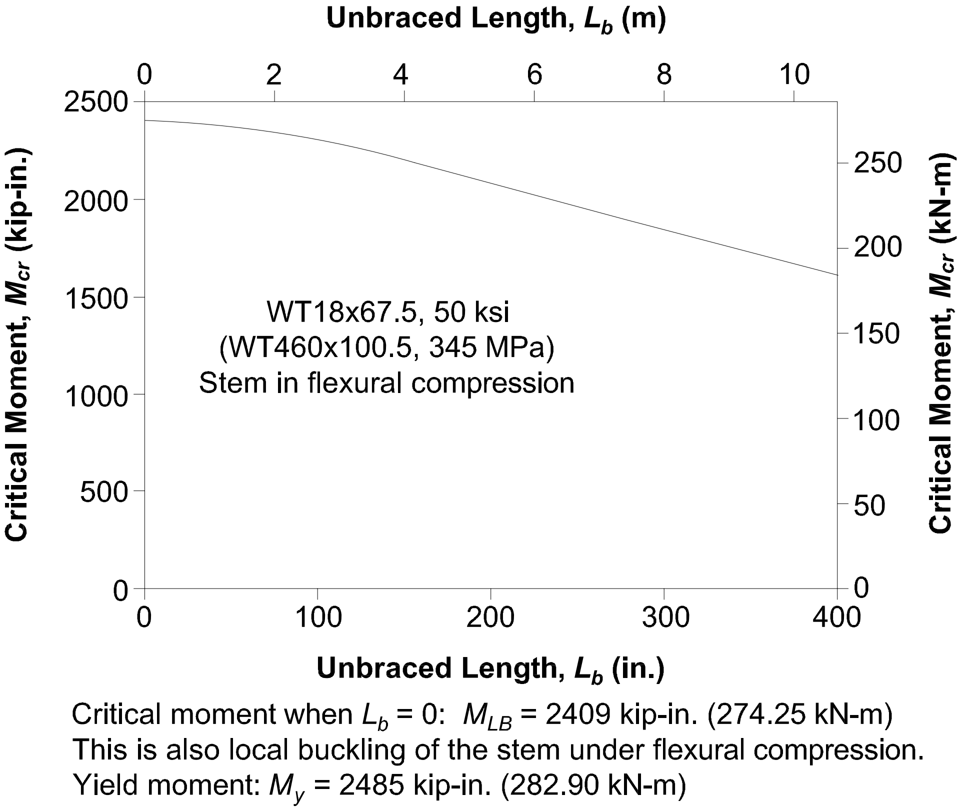


Fig. C-F9.1. Critical moment for a tee beam [WT18×67.5 (WT460×100.5), $F_y = 50$ ksi (345 MPa)].

may be used in Equations F10-2 or F10-3 to obtain the nominal flexural strength.

F10. SINGLE ANGLES

Flexural strength limits are established for the limit states of yielding, local buckling and lateral-torsional buckling of single-angle beams. In addition to addressing the general case of unequal-leg single angles, the equal-leg angle is treated as a special case. Furthermore, bending of equal-leg angles about a geometric axis, an axis parallel to one of the legs, is addressed separately as it is a common case of angle bending.

The tips of an angle refer to the free edges of the two legs. In most cases of unrestrained bending, the flexural stresses at the two tips will have the same sign (tension or compression). For constrained bending about a geometric axis, the tip stresses will differ in sign. Provisions for both tension and compression at the tip should be checked as appropriate, but in most cases it will be evident which controls.

Appropriate serviceability limits for single-angle beams need also to be considered. In particular, for longer members subjected to unrestrained bending, deflections are likely to control rather than lateral-torsional or local buckling strength.

The provisions in this section follow the general format for nominal flexural resistance (see Figure C-F1.2). There is a region of full yielding, a linear transition to the yield moment, and a region of local buckling.

1. Yielding

The strength at full yielding is limited to a shape factor of 1.50 applied to the yield moment. This leads to a lower bound plastic moment for an angle that could be bent about any axis, inasmuch as these provisions are applicable to all flexural conditions. The 1.25 factor originally used was known to be a conservative value. Recent research work (Earls and Galambos, 1997) has indicated that the 1.50 factor represents a better lower bound value. Since the shape factor for angles is in excess of 1.50, the nominal design strength $M_n = 1.5M_y$ for compact members is justified provided that instability does not control.

2. Lateral-Torsional Buckling

Lateral-torsional buckling may limit the flexural strength of an unbraced single-angle beam. As illustrated in Figure C-F10.1, Equation F10-2 represents the elastic buckling portion with the maximum nominal flexural strength, M_n, equal to 75 percent of the theoretical buckling moment, M_e. Equation F10-3 represents the inelastic buckling transition expression between $0.75M_y$ and $1.5M_y$. The maximum beam flexural strength $M_n = 1.5M_y$ will occur when the theoretical buckling moment, M_e, reaches or exceeds $7.7M_y$. These equations are modifications of those developed from the results of Australian research on single angles in flexure and on an analytical model consisting of two rectangular elements of length equal to the actual angle leg width minus one-half the thickness (AISC, 1975; Leigh and Lay, 1978; Leigh and Lay, 1984; Madugula and Kennedy, 1985).

When bending is applied about one leg of a laterally unrestrained single angle, the angle will deflect laterally as well as in the bending direction. Its behavior can be evaluated by resolving the load and/or moments into principal axis components and determining the sum of these principal axis flexural effects. Section F10.2(i) is provided to simplify and expedite the calculations for this common situation with equal-leg angles.

For such unrestrained bending of an equal-leg angle, the resulting maximum normal stress at the angle tip (in the direction of bending) will be approximately 25 percent greater than the calculated stress using the geometric axis section modulus. The value of M_e in Equation F10-5 and the evaluation of M_y using 0.80 of the geometric axis section modulus reflect bending about the inclined axis shown in Figure C-F10.2.

The deflection calculated using the geometric axis moment of inertia has to be increased 82 percent to approximate the total deflection. Deflection has two components, a vertical component (in the direction of applied load) 1.56 times the calculated value and a horizontal component of 0.94 times the calculated value. The resultant total deflection is in the general direction of the weak principal axis bending of the angle (see Figure C-F10.2). These unrestrained bending deflections should be considered in evaluating serviceability and will often control the design over lateral-torsional buckling.

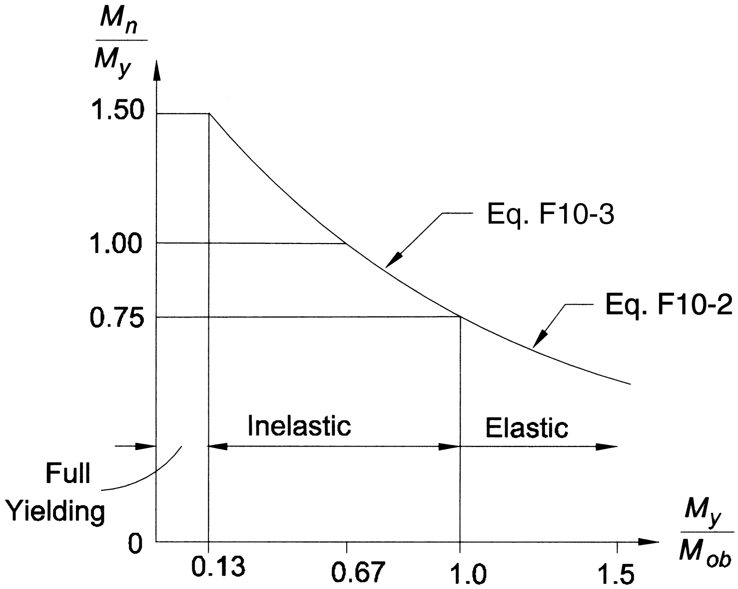


Fig. C-F10.1. Lateral-torsional buckling limits of a single-angle beam.

The horizontal component of deflection being approximately 60 percent of the vertical deflection means that the lateral restraining force required to achieve purely vertical deflection must be 60 percent of the applied load value (or produce a moment 60 percent of the applied value) which is very significant.

Lateral-torsional buckling is limited by M_e (Leigh and Lay, 1978; Leigh and Lay, 1984) in Equation F10-4a, which is based on

$$M_{cr} = \frac{2.33Eb^4t}{(1+3\cos^2\theta)(Kl)^2} \times \left[\sqrt{\sin^2\theta + \frac{0.156(1+3\cos^2\theta)(Kl)^2t^2}{b^4}} + \sin\theta\right] \qquad \text{(C-F10-1)}$$

(the general expression for the critical moment of an equal-leg angle) with $\theta = 45°$ or the condition where the angle tip stress is compressive (see Figure C-F10.3). Lateral-torsional buckling can also limit the flexural strength of the cross section when the maximum angle tip stress is tensile from geometric axis flexure, especially with use of the flexural strength limits in Section F10.2. Using $\theta = 45°$ in Equation C-F10-1, the resulting expression is Equation F10-4b with a $+1$ instead of -1 as the last term.

Stress at the tip of the angle leg parallel to the applied bending axis is of the same sign as the maximum stress at the tip of the other leg when the single angle is unrestrained. For an equal-leg angle this stress is about one-third of the maximum stress. It is only necessary to check the nominal bending strength based on the tip of the angle leg with the maximum stress when evaluating such an angle. Since this maximum moment per Section F10.2(ii) represents combined principal axis moments and Equation F10-5 represents the design limit for these

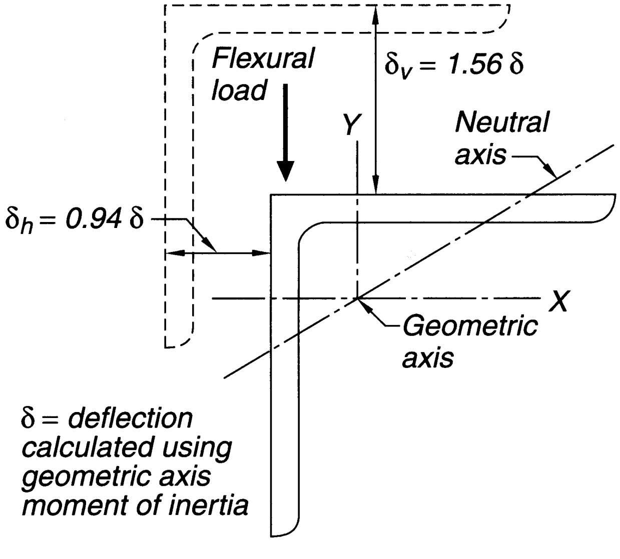


Fig. C-F10.2. Geometric axis bending of laterally unrestrained equal-leg angles.

combined flexural moments, only a single flexural term needs to be considered when evaluating combined flexural and axial effects.

For unequal-leg angles without lateral-torsional restraint, the applied load or moment must be resolved into components along the two principal axes in all cases and design must be for *biaxial bending* using the interaction equations in Chapter H.

Under major axis bending of equal-leg angles, Equation F10-5 in combination with Equations F10-2 and F10-3 controls the available moment against overall lateral-torsional buckling of the angle. This is based on M_e, given earlier with $\theta = 0$.

Lateral-torsional buckling for this case will reduce the stress below $1.5M_y$ only for $l/t \geq 3675C_b/F_y(M_e = 7.7M_y)$. If the lt/b^2 parameter is small (less than approximately $0.87C_b$ for this case), local buckling will control the available moment and M_n based on lateral-torsional buckling need not be evaluated. Local buckling must be checked using Section F10.3.

Lateral-torsional buckling about the major principal w-axis of an unequal-leg angle is controlled by M_e in Equation F10-6. The section property β_w reflects the location of the shear center relative to the principal axis of the section and the bending direction under uniform bending. Positive β_w and maximum M_e occurs when the shear center is in flexural compression while negative β_w and minimum M_e occur when the shear center is in flexural tension (see Figure C-F10.4). This β_w effect is consistent with behavior of singly symmetric I-shaped beams, which are more stable when the compression flange is larger than the tension flange. For principal w-axis bending of equal-leg angles, β_w is equal to zero due to symmetry and Equation F10-6 reduces to Equation F10-5 for this special case.

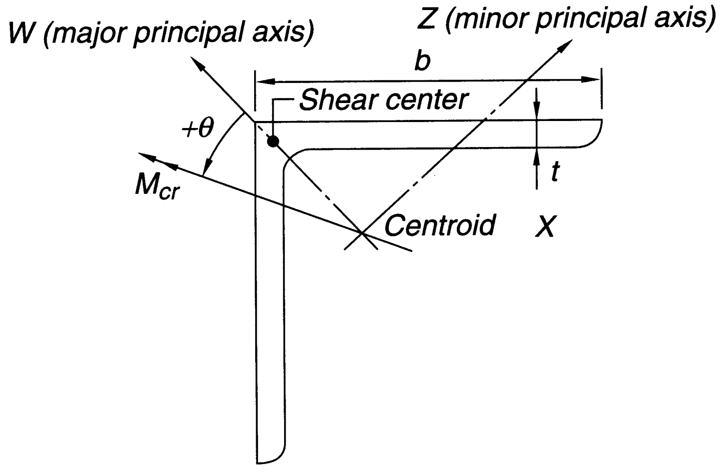


Fig. C-F10.3. Equal-leg angle with general moment loading.

For reverse curvature bending, part of the unbraced length has positive β_w, while the remainder has negative β_w; conservatively, the negative value is assigned for that entire unbraced segment.

The factor β_w is essentially independent of angle thickness (less than one percent variation from mean value) and is primarily a function of the leg widths. The average values shown in Table C-F10.1 may be used for design.

3. Leg Local Buckling

The b/t limits have been modified to be more representative of flexural limits rather than using those for single angles under uniform compression. Typically the flexural stresses will vary along the leg length permitting the use of the stress limits given. Even for the geometric axis flexure case, which produces uniform compression along one leg, use of these limits will provide a conservative value when compared to the results reported in Earls and Galambos (1997).

F11. RECTANGULAR BARS AND ROUNDS

The provisions in Section F11 apply to solid bars with round and rectangular cross section. The prevalent limit state for such members is the attainment of the full plastic moment, M_p. The exception is the lateral-torsional buckling of rectangular bars where the depth is larger than the width. The requirements for design are identical to those given previously in Table A-F1.1 in the 1999 *LRFD Specification* (AISC, 2000b). Since the shape factor for a rectangular cross section is 1.5 and for a round section is 1.7, consideration must be given to serviceability issues such as excessive deflection or permanent deformation under service-load conditions.

F12. UNSYMMETRICAL SHAPES

When the design engineer encounters beams that do not contain an axis of symmetry, or any other shape for which there are no provisions in the other sections of Chapter F, the stresses are to be limited by the yield stress or the elastic buckling

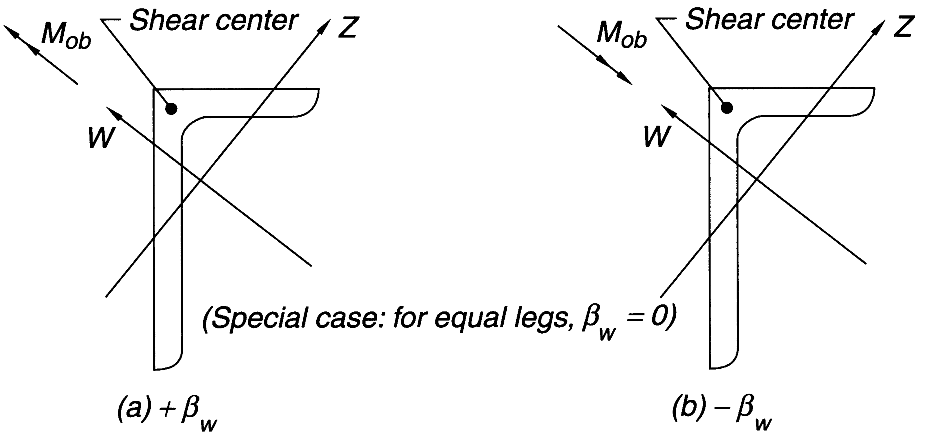


Fig. C-F10.4. Unequal-leg angle in bending.

TABLE C-F10.1
β_w Values for Angles

Angle Size [in. (mm)]	β_w [in. (mm)]*
9 × 4 (229 × 102)	6.54 (116)
8 × 6 (203 × 152)	3.31 (84.1)
8 × 4 (203 × 102)	5.48 (139)
7 × 4 (178 × 102)	4.37 (111)
6 × 4 (152 × 102)	3.14 (79.8)
6 × 3.5 (152 × 89)	3.69 (93.7)
5 × 3.5 (127 × 89)	2.40 (61.0)
5 × 3 (127 × 76)	2.99 (75.9)
4 × 3.5 (102 × 89)	0.87 (22.1)
4 × 3 (102 × 76)	1.65 (41.9)
3.5 × 3 (89 × 76)	0.87 (22.1)
3.5 × 2.5 (89 × 64)	1.62 (41.1)
3 × 2.5 (76 × 64)	0.86 (21.8)
3 × 2 (76 × 51)	1.56 (39.6)
2.5 × 2 (64 × 51)	0.85 (21.6)
Equal legs	0.00

$^*\beta_w = \dfrac{1}{I_w}\int_A z\left(w^2 + z^2\right) dA - 2z_o$ where z_o is the coordinate along the z-axis of the shear center with respect to the centroid, and I_w is the moment of inertia for the major principal axis; β_w has positive or negative value depending on direction of bending (see Figure C-F10.4)

stress. The stress distribution and/or the elastic buckling stress must be determined from principles of structural mechanics, text books or handbooks, such as the SSRC Guide (Galambos, 1998), papers in journals, or finite element analyses. Alternatively, the designer can avoid the problem by selecting cross sections from among the many choices given in the previous sections of Chapter F.

F13. PROPORTIONS OF BEAMS AND GIRDERS

1. Hole Reductions

Historically, provisions for proportions of rolled beams and girders with holes in the tension flange were based upon either a percentage reduction independent of material strength or a calculated relationship between the tension rupture and tension yield strengths of the flange, with resistance factors or safety factors included in the calculation. In both cases, the provisions were developed based upon tests of steel with a specified minimum yield stress of 36 ksi (248 MPa) or less.

More recent tests (Dexter and Altstadt, 2004; Yuan, Swanson, and Rassati, 2004) indicate that the flexural strength on the net section is better predicted by comparison of the quantities $F_y A_{fg}$ and $F_u A_{fn}$, with slight adjustment when the ratio of F_y to F_u exceeds 0.8. If the holes remove enough material to affect the member strength, the critical stress is adjusted from F_y to $(F_u A_{fn}/A_{fg})$ and this value is conservatively applied to the elastic section modulus S_x.

2. Proportioning Limits for I-Shaped Members

The provisions of this section are taken directly from Appendix G, Section G1 of the 1999 *LRFD Specification* (AISC, 2000b). They have been part of the plate-girder design requirements since 1963; they are derived from Basler and Thurlimann (1963). The web depth-thickness limitations are provided so as to prevent the flange from buckling into the web. Equation F13-4 is slightly modified from the corresponding Equation A-G1-2 in the 1999 Specification to recognize the change in this Specification in the definition of *residual stress* from a flat 16.5 ksi (114 MPa) used previously to 30 percent of the yield stress, as shown by the following derivation,

$$\frac{0.48E}{\sqrt{F_y\left(F_y+16.5\right)}} \approx \frac{0.48E}{\sqrt{F_y\left(F_y+0.3F_y\right)}} = \frac{0.42E}{F_y} \qquad \text{(C-F13-1)}$$

CHAPTER G

DESIGN OF MEMBERS FOR SHEAR

G1. GENERAL PROVISIONS

Chapter G applies to webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and HSS, and shear in the weak direction of singly or doubly symmetric shapes.

Two methods for determining the shear strength of singly or doubly symmetric I-shaped beams and built-up sections are presented. The method of Section G2 does not utilize the post-buckling strength of the web, while the method of Section G3 utilizes the post-buckling strength.

G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

Section G2 deals with the shear strength of webs of wide-flange or I-shaped members, as well as webs of tee-shapes, that are subject to shear and bending in the plane of the web. The provisions in Section G2 apply to the general case when an increase of strength due to tension field action is not permitted. Conservatively, these provisions may be applied also when it is not desired to use the tension field action enhancement for convenience in design. Consideration of the effect of bending on the shear strength is not required because the effect is deemed negligible.

1. Nominal Shear Strength

The nominal shear strength of a web is defined by Equation G2-1, a product of the shear yield force $0.6F_yA_w$ and the shear-buckling reduction factor C_v.

The provisions of case (a) in Section G2.1 for rolled I-shaped members with $h/t_w \leq 2.24\sqrt{E/F_y}$ are similar to previous LRFD provisions, with the exception that ϕ has been increased from 0.90 to 1.00 (with a corresponding decrease of the safety factor from 1.67 to 1.5), thus making these provisions consistent with previous provisions for allowable stress design. The value of ϕ of 0.90 is justified by comparison with experimental test data and recognizes the minor consequences of shear yielding, as compared to those associated with tension and compression yielding, on the overall performance of rolled I-shaped members. This increase is applicable only to the shear yielding limit state of I-shaped members.

Case (b) in Section G2.1 uses the shear buckling reduction factor, C_v, shown in Figure C-G2.1. The curve for C_v has three segments.

For webs with $h/t_w \le 1.10\sqrt{Ek_v/F_y}$, the nominal shear strength V_n is based on shear yielding of the web, with C_v given by Equation G2-3. This h/t_w limit was determined by setting the critical stress causing shear buckling, F_{cr}, equal to the yield stress of the web, $F_{yw} = F_y$, in Equation 35 of Cooper, Galambos, and Ravindra (1978).

When $h/t_w > 1.10\sqrt{Ek_v/F_y}$, the web shear strength is based on buckling. It has been suggested to take the proportional limit as 80 percent of the yield stress of the web (Basler, 1961). This corresponds to $h/t_w = (1.10/0.8)\left(\sqrt{Ek_v/F_y}\right)$.

When $h/t_w > 1.37\sqrt{Ek_v/F_y}$, the web strength is determined from the elastic buckling stress given by Equation 6 of Cooper and others (1978) and Equation 9-7 in Timoshenko and Gere (1961):

$$F_{cr} = \frac{\pi^2 E k_v}{12\left(1-\nu^2\right)\left(h/t_w\right)^2} \tag{C-G2-1}$$

C_v in Equation G2-5 was obtained by dividing F_{cr} from Equation C-G2-1 by $0.6F_yA_w$ and using $E = 29{,}000$ ksi (200 000 MPa) and $\nu = 0.3$.

A straight-line transition for C_v (Equation G2-4) is used between the limits given by $1.10\sqrt{k_vE/F_y} < h/t_w \le 1.37\sqrt{k_vE/F_y}$.

The plate buckling coefficient, k_v, for panels subject to pure shear having simple supports on all four sides is given by Equation 4.24 in Galambos (1998).

$$k_v = \left\{ \begin{array}{ll} 4.00 + \dfrac{5.34}{(a/h)^2} & \text{for } a/h \le 1 \\ 5.34 + \dfrac{4.00}{(a/h)^2} & \text{for } a/h > 1 \end{array} \right\} \tag{C-G2-2}$$

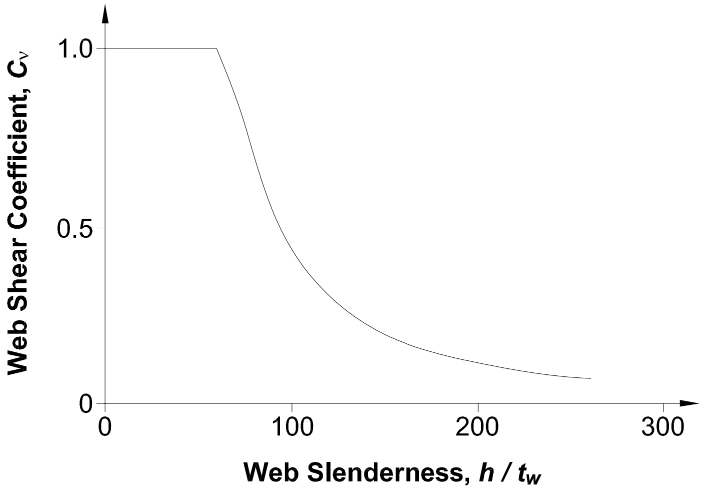


Fig. C-G2.1. Shear buckling coefficient C_v for $F_y = 50$ ksi (345 MPa) and $k_v = 5.0$.

For practical purposes and without loss of accuracy, these equations have been simplified herein and in AASHTO (1998) to

$$k_v = 5 + \frac{5}{(a/h)^2} \tag{C-G2-3}$$

When the panel ratio a/h becomes large, as in the case of webs without transverse stiffeners, then $k_v = 5$. Equation C-G2-3 applies as long as there are flanges on both edges of the web. For tee-shaped beams the free edge is unrestrained and for this situation $k_v = 1.2$ (JCRC, 1971).

The provisions of Section G2.1 assume monotonically increasing loads. If a flexural member is subjected to load reversals causing cyclic yielding over large portions of a web, such as may occur during a major earthquake, special design considerations may apply (Popov, 1980).

2. Transverse Stiffeners

When transverse stiffeners are needed, they must be rigid enough to cause a buckling node to form along the line of the stiffener. This requirement applies whether or not tension field action is counted upon. The required moment of inertia of the stiffener is the same as in AASHTO (1996), but it is different from the formula $I_{st} \geq (h/50)^4$ in the 1989 *ASD Specification* (AISC, 1989). Equation G2-5 is derived in Chapter 11 of Salmon and Johnson (1996). The origin of the formula can be traced to Bleich (1952).

G3. TENSION FIELD ACTION

The provisions of Section G3 apply when it is intended to account for the enhanced strength of webs of built-up members due to tension field action.

1. Limits on the Use of Tension Field Action

The panels of the web of a built-up member, bounded on top and bottom by the flanges and on each side by the transverse stiffeners, are capable of carrying loads far in excess of their “web buckling” load. Upon reaching the theoretical web buckling limit, very slight lateral web displacements will have developed. These deformations are of no structural significance, because other means are still present to provide further strength.

When transverse stiffeners are properly spaced and are strong enough to act as compression struts, membrane stresses due to shear forces greater than those associated with the theoretical web buckling load form diagonal tension fields in the web panels. The resulting combination in effect provides a Pratt truss that furnishes the strength to resist applied shear forces unaccounted for by the linear buckling theory.

The key point in the development of tension field action in the web of plate girders is the ability of the stiffeners to support the compression from the two panels on either side of the stiffener. In the case of end panels there is a panel only on one side. The support of the tension field forces is also reduced when the panel aspect ratio becomes too large. For this reason the inclusion of the tension field enhancement is not permitted for end panels and when *a/h* exceeds 3.0 or $\left[\frac{260}{(h/t_w)}\right]^2$.

2. Nominal Shear Strength with Tension Field Action

Analytical methods based on tension field action have been developed (Basler and Thurlimann, 1963; Basler, 1961) and corroborated in an extensive program of tests (Basler, Yen, Mueller, and Thurlimann, 1960). Equation G3-2 is based on this research. The second term in the bracket represents the relative increase of the panel shear strength due to tension field action.

3. Transverse Stiffeners

The vertical component of the tension field force that is developed in the web panel must be resisted by the transverse stiffener. In addition to the rigidity required to keep the line of the stiffener as a nonmoving point for the buckled panel, as provided for in Section G2.2, the stiffener must also have a large enough area to resist the tension field reaction. Equation G3-3 often controls the design of the stiffeners.

G4. SINGLE ANGLES

Shear stresses in single-angle members are the result of the gradient of the bending moment along the length (flexural shear) and the torsional moment.

The maximum elastic stress due to flexural shear is

$$f_v = \frac{1.5V_b}{bt} \tag{C-G4-1}$$

where V_b is the component of the shear force parallel to the angle leg with width b and thickness t. The stress is constant throughout the thickness, and it should be calculated for both legs to determine the maximum. The coefficient 1.5 is the calculated value for equal leg angles loaded along one of the principal axes. For equal leg angles loaded along one of the geometric axes, this factor is 1.35. Factors between these limits may be calculated conservatively from V_bQ/It to determine the maximum stress at the neutral axis. Alternatively, if only flexural shear is considered, a uniform flexural shear stress in the leg of V_b/bt may be used due to inelastic material behavior and stress redistribution.

If the angle is not laterally braced against twist, a torsional moment is produced equal to the applied transverse load times the perpendicular distance *e* to the shear center, which is at the point of intersection of the centerlines of the two legs. Torsional moments are resisted by two types of shear behavior: pure torsion (*St. Venant torsion*) and *warping torsion* [see Seaburg and Carter (1997)]. The shear stresses due to restrained warping are small compared to the *St. Venant torsion* (typically less than 20 percent) and they can be neglected for practical purposes. The applied torsional moment is then resisted by pure shear stresses that are constant along the width of the leg (except for localized regions at the toe of the leg), and the maximum value can be approximated by

$$f_v = \frac{M_T t}{J} = \frac{3M_T}{At} \qquad \text{(C-G4-2)}$$

where

J = torsional constant (approximated by $\Sigma(bt^3/3)$ when precomputed value is unavailable)

A = angle cross-sectional area

For a study of the effects of warping, see Gjelsvik (1981). Torsional moments from laterally unrestrained transverse loads also produce warping normal stresses that are superimposed on the bending stresses. However, since the warping strength of single angles is relatively small, this additional bending effect, just like the warping shear effect, can be neglected for practical purposes.

G5. RECTANGULAR HSS AND BOX MEMBERS

The two webs of a closed-section rectangular cross section resist shear the same way as the single web of an I-shaped plate girder or wide-flange beam, and therefore, the provisions of Section G2 apply.

G6. ROUND HSS

Little information is available on round HSS subjected to transverse shear and the recommendations are based on provisions for local buckling of cylinders due to torsion. However, since torsion is generally constant along the member length and transverse shear usually has a gradient; it is recommended to take the critical stress for transverse shear as 1.3 times the critical stress for torsion (Brockenbrough and Johnston, 1981; Galambos, 1998). The torsion equations apply over the full length of the member, but for transverse shear it is reasonable to use the length between the points of maximum and zero shear force. Only thin HSS may require a reduction in the shear strength based upon first shear yield. Even in this case, shear will only govern the design of round HSS for the case of thin sections with short spans.

In the equation for the nominal shear strength, V_n, of round HSS, it is assumed that the shear stress at the neutral axis, calculated as VQ/Ib, is at F_{cr}. For a thin round section with radius R and thickness t, $I = \pi R^3 t$, $Q = 2R^2 t$ and $b = 2t$. This gives the stress at the centroid as $V/\pi Rt$, in which the denominator is recognized as half the area of the round HSS.

G7. WEAK AXIS SHEAR IN SINGLY AND DOUBLY SYMMETRIC SHAPES

The nominal shear strength of singly and doubly symmetric I-shapes is governed by the equations of Section G2 with the plate buckling coefficient equal to $k_v = 1.2$, the same as the web of a tee-shape. The maximum plate slenderness of all rolled shapes is $(b_f/2t_f) = 13.8$, and for $F_y = 100$ ksi (690 MPa) the value of $1.10\sqrt{\dfrac{k_v E}{F_y}} = 1.10\sqrt{\dfrac{1.2 \times 29000}{100}} = 20.5$. Thus $C_v = 1.0$ except for built-up shapes with very slender flanges.

G8. BEAMS AND GIRDERS WITH WEB OPENINGS

Web openings in structural floor members may be used to accommodate various mechanical, electrical and other systems. Strength limit states, including local buckling of the compression flange or of the web, local buckling or yielding of the tee-shaped compression zone above or below the opening, lateral buckling and moment-shear interaction, or serviceability may control the design of a flexural member with web openings. The location, size and number of openings are important and empirical limits for them have been identified. One general procedure for assessing these effects and the design of any needed reinforcement for both steel and composite beams is given in the ASCE *Specification for Structural Steel Beams with Web Openings* (ASCE, 1999), with background information provided in Darwin (1990) and in ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992) and ASCE Task Committee on Design Criteria for Composite Structures in Steel and Concrete (1992a).

CHAPTER H

DESIGN OF MEMBERS FOR COMBINED FORCES AND TORSION

Chapters D, E, F and G of this Specification address members subject to only one type of force: axial tension, axial compression, flexure and shear, respectively. Chapter H addresses members subject to a combination of two or more of the individual forces defined above, as well as possibly by additional forces due to torsion. The provisions fall into two categories: (a) the majority of the cases that can be handled by an interaction equation involving sums of ratios of required strengths to the available strengths; and (b) cases where the stresses due to the applied forces are added and compared to limiting buckling or yield stresses. Designers will have to consult the provisions of Sections H2 and H3 only in rarely occurring cases.

H1. DOUBLY AND SINGLY SYMMETRIC MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

1. Doubly and Singly Symmetric Members in Flexure and Compression

Section H1 contains design provisions for prismatic members under combined flexure and compression and under combined flexure and tension for doubly and singly symmetric sections. The provisions of Section H1 apply typically to rolled wide-flange shapes, channels, tee-shapes, round, square and rectangular HSS, solid rounds, squares, rectangles or diamonds, and any of the many possible combinations of doubly or singly symmetric shapes fabricated from plates and/or shapes by welding or bolting. The interaction equations accommodate flexure about one or both principal axes as well as axial compression or tension.

In 1923, the first AISC Specification required that the stresses due to flexure and compression be added and that the sum not exceed the allowable value. An interaction equation appeared first in the 1936 Specification, stating “Members subject to both axial and bending stresses shall be so proportioned that the quantity $\frac{f_a}{F_a} + \frac{f_b}{F_b}$ shall not exceed unity,” in which F_a and F_b are, respectively, the axial and flexural allowable stresses permitted by this Specification, and f_a and f_b are the corresponding stresses due to the axial force and the bending moment, respectively. This linear interaction equation was in force until the 1961 Specification, when it was modified to account for frame stability and for the P-δ effect, that is, the secondary bending between the ends of the members (Equation C-H1-1). The P-Δ effect, that is, the second-order bending moment due to story sway, was not accommodated.

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F'_e}\right) F_b} \le 1.0 \tag{C-H1-1}$$

The allowable axial stress F_a is determined for an effective length that is larger than unity for moment frames. The term $\frac{1}{1-\frac{f_a}{F'_e}}$ is the amplification of the interspan moment due to member deflection multiplied by the axial force (the P-δ effect). C_m accounts for the effect of the moment gradient. This interaction equation has been part of all the subsequent editions of the AISC ASD Specifications since 1961.

A new approach to the interaction of flexural and axial forces was introduced in the 1986 AISC *LRFD Specification* (AISC, 1986). The following is an explanation of the thinking behind the interaction curves used. The equations

$$\begin{aligned} \frac{P}{P_y} + \frac{8}{9}\frac{M_{pc}}{M_p} &= 1 \quad \text{for } \frac{P_u}{P_y} \ge 0.2 \\ \frac{P}{2P_y} + \frac{M_{pc}}{M_p} &= 1 \quad \text{for } \frac{P_u}{P_y} < 0.2 \end{aligned} \tag{C-H1-2}$$

define the lower-bound curve for the interaction of the nondimensional axial strength P/P_y and flexural strength M/M_p for compact wide-flange stub-columns bent about their x-axis. The cross section is assumed to be fully yielded in tension and compression. The symbol M_{pc} is the plastic moment strength of the cross section in the presence of an axial force P. The curve representing Equation C-H1-2 almost overlaps the analytically exact curve for the major-axis bending of a W8×31 (W200×46.1) cross section (see Figure C-H1.1). The equations for the exact yield capacity of a wide-flange shape are (ASCE, 1971):

$$\begin{aligned} &\text{for } 0 \le \frac{P}{P_y} \le \frac{t_w\left(d - 2t_f\right)}{A} \\ &\qquad \frac{M_{pc}}{M_p} = 1 - \frac{A^2\left(\frac{P}{P_y}\right)^2}{4t_w Z_x} \\ &\text{for } \frac{t_w\left(d-2t_f\right)}{A} \le \frac{P}{P_y} \le 1 \\ &\qquad \frac{M_{pc}}{M_p} = \frac{A\left(1-\frac{P}{P_y}\right)}{2Z_x}\left[d - \frac{A\left(1-\frac{P}{P_y}\right)}{2b_f}\right] \end{aligned} \tag{C-H1-3}$$

The equation approximating the average yield strength of wide-flange shapes is

$$\frac{M_{pc}}{M_p} = 1.18\left(1 - \frac{P}{P_y}\right) \le 1 \tag{C-H1-4}$$

The curves in Figure C-H1.2 show the exact and approximate yield interaction curves for wide-flange shapes bent about the y-axis, and the exact curves for the solid rectangular and round shapes. It is evident that the lower-bound AISC interaction curves are very conservative for these shapes.

The idea of portraying the strength of stub beam-columns was extended to actual beam-columns with actual lengths by normalizing the required flexural strength, M_u, of the beam by the nominal strength of a beam without axial force, M_n, and the required axial strength, P_u, by the nominal strength of a column without bending moment, P_n. This rearrangement results in a translation and rotation of the original stub-column interaction curve, as seen in Figure C-H1.3.

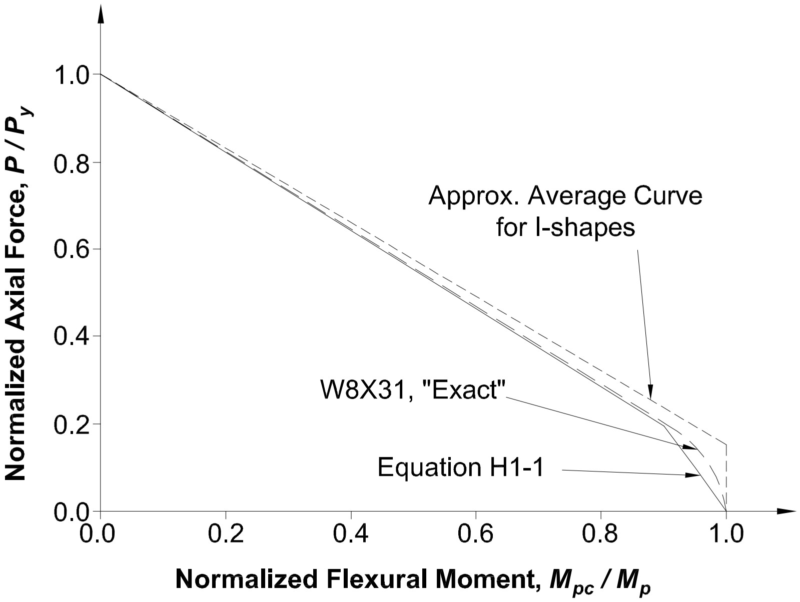


Fig. C-H1.1. Stub-column interaction curves: plastic moment versus axial force for wide-flange shapes, major-axis flexure [W8×31 (W200×46.1), F_y = 50 ksi (345 MPa)].

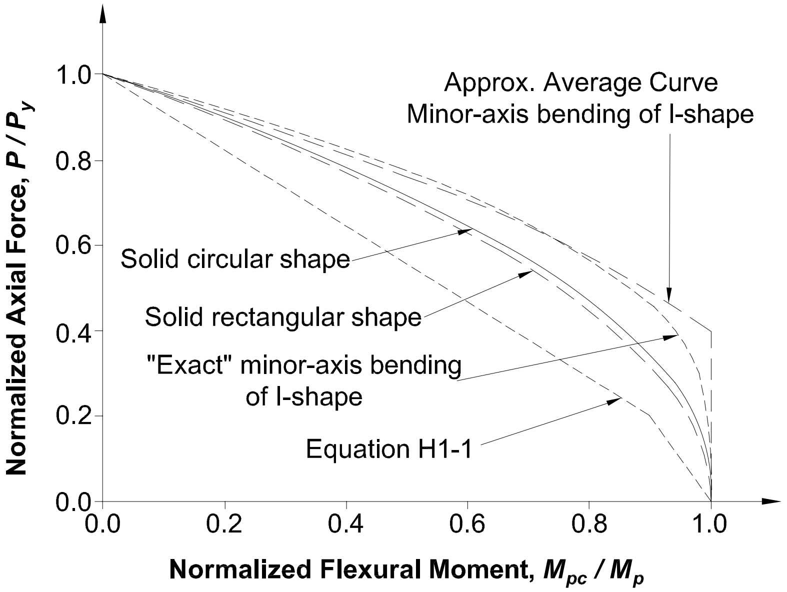


Fig. C-H1.2. Stub-column interaction curves: plastic moment versus axial force for solid round and rectangular sections and for wide-flange shapes, minor-axis flexure.

The normalized equations corresponding to the beam-column with length effects included are shown as Equation C-H1-5:

$$\frac{P_u}{P_n} + \frac{8}{9}\frac{M_u}{M_n} = 1 \text{ for } \frac{P_u}{P_n} \geq 0.2$$

$$\frac{P_u}{2P_n} + \frac{M_u}{M_n} = 1 \text{ for } \frac{P_u}{P_n} < 0.2 \qquad \text{(C-H1-5)}$$

The interaction equations are designed to be very versatile. The terms in the denominator fix the endpoints of the interaction curve. The nominal flexural strength, M_n, is determined by the appropriate provisions from Chapter F. It encompasses the limit states of yielding, lateral-torsional buckling, flange local buckling and web local buckling.

The axial term, P_n, is governed by the provisions of Chapter E, and it can accommodate compact or slender columns, as well as the limit states of major and minor axis buckling, and torsional and flexural-torsional buckling. Furthermore, P_n is calculated for the applicable effective length of the column to take care of frame stability effects, if the procedures of Section C.2-1a and Section C.2-1b are used to determine the required moments and axial forces. These moments and axial forces include the amplification due to second-order effects.

The utility of the interaction equations is further enhanced by the fact that they also permit the consideration of *biaxial bending*.

2. Doubly and Singly Symmetric Members in Flexure and Tension

Section H1.1 considers the most frequently occurring cases in design: members under flexure and axial compression. Section H1.2 addresses the less frequent cases of flexure and axial tension. Since axial tension increases the bending

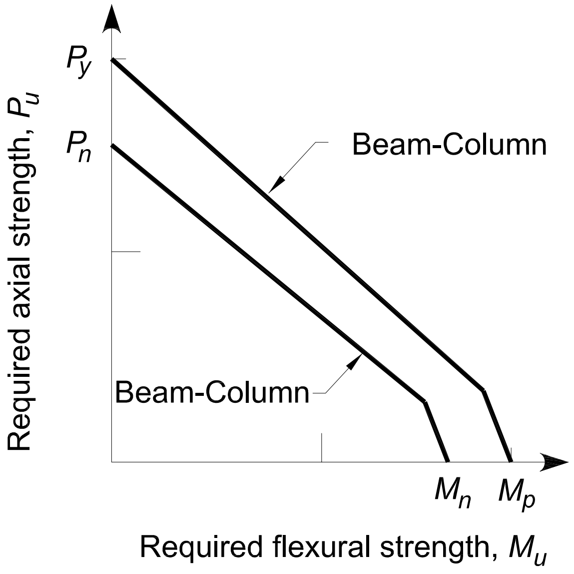


Fig. C-H1.3. Interaction curve for stub beam-column and beam-column.

stiffness of the member to some extent, Section H1.2 permits the increase of the bending terms in the interaction equations in proportion to $\sqrt{1+\dfrac{P_u}{P_{ey}}}$.

3. Doubly Symmetric Members in Single Axis Flexure and Compression

The linear interaction Equation C-H1-5 is conservative for cases where the axial limit state is out-of-plane buckling and the flexural limit state is lateral-torsional buckling for doubly symmetric wide-flange sections with moment applied about the x-axis (Galambos, 1998). Section H1.3 gives an optional equation for such beam-columns.

The two curves in Figure C-H1.4 illustrate the difference between the bi-linear and the parabolic interaction equations for the case of a W27×84 (W690×125) beam-column.

The relationship between Equations H1-1 and H1-2 is further illustrated in Figures C-H1.5 (for LRFD) and C-H1.6 (for ASD). The curves relate the required axial force, P (ordinate), and the required bending moment, M (abscissa), when the interaction Equations H1-1 and H1-2 are equal to unity. The positive values of P are compression and the negative values are tension. The curves are for a 10 ft (3 m) long W16×26 [$F_y = 50$ ksi (345 MPa)] member. The solid curve is for in-plane behavior, that is, lateral bracing prevents lateral-torsional buckling. The dotted curve represents Equation H1-1 for the case when there are no lateral braces between the ends of the beam-column. In the region of the tensile axial force, the curve is modified by the term $\sqrt{1+\dfrac{P}{P_y}}$, as permitted in Section H1.2. The dashed

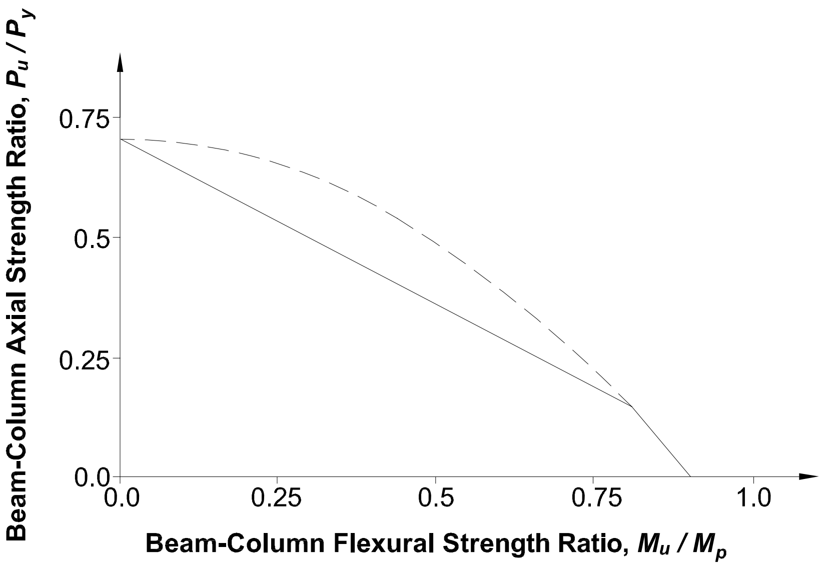


Fig. C-H1.4. Comparison between bi-linear (Equation H1-1) and parabolic (Equation H1-2) interaction equations [W27×84 (W690×125), $F_y = 50$ ksi (345 MPa), $L_b = 10$ ft (3.05 m), $C_b = 1.75$].

curve is Equation H1-2. For a given compressive or tensile axial force, the latter equation allows a larger bending moment over most of its domain of applicability.

H2. UNSYMMETRIC AND OTHER MEMBERS SUBJECT TO FLEXURE AND AXIAL FORCE

The provisions of Section H1 apply to beam-columns with cross sections that are either doubly or singly symmetric. However, there are many cross sections that are unsymmetrical, such as unequal leg angles and any number of possible fabricated sections. For these situations the interaction equation of Section H1 may not be appropriate. The linear interaction $\frac{f_a}{F_a} + \frac{f_{bw}}{F_{bw}} + \frac{f_{bz}}{F_{bz}} \leq 1.0$ provides a conservative and simple way to deal with such problems. The lower case stresses f are the required axial and flexural stresses computed by elastic analysis for the applicable loads, including second-order effects where appropriate, and the upper case stresses F are the available stresses corresponding to the limit state of yielding or buckling. The subscripts w and z refer to the principal axes of the unsymmetric cross section. This Specification leaves the option to the designer to use the Section H2 interaction equation for cross sections that would qualify for the more liberal interaction equation of Section H1.

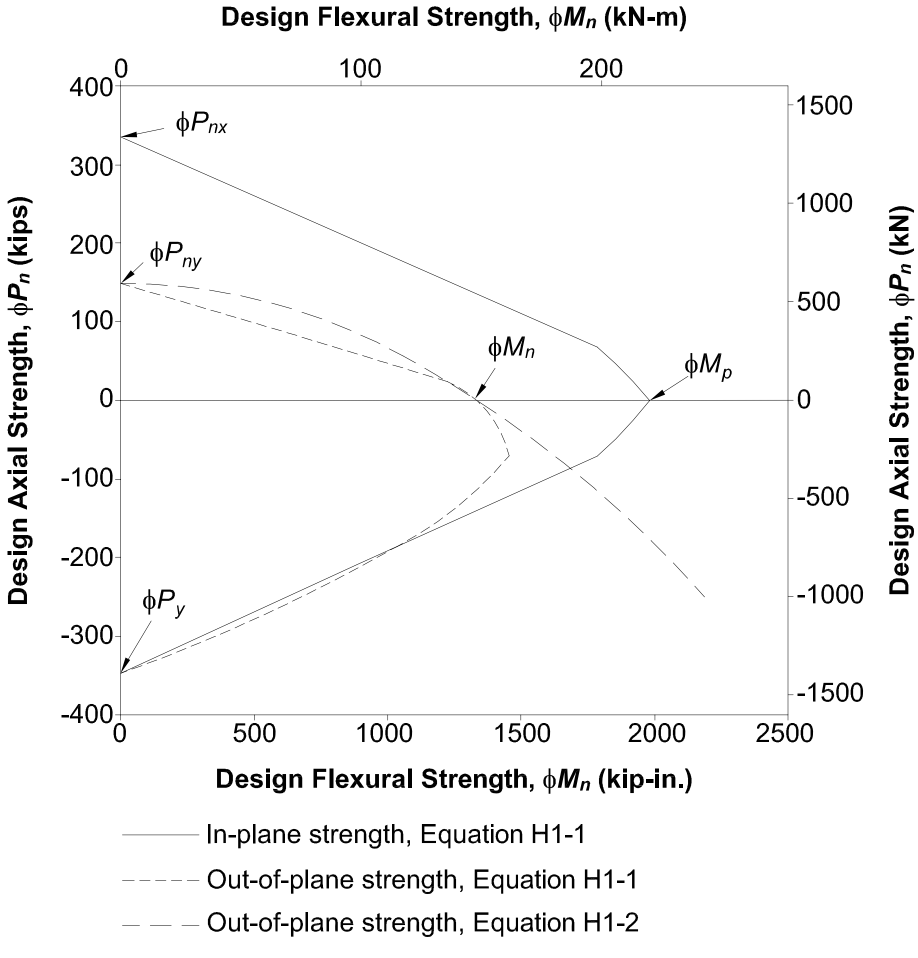


Fig. C-H1.5. Beam-columns under compressive and tensile axial force (tension is shown as negative) (LRFD) [W16×26 (W410×38.8), $F_y = 50$ ksi (345 MPa), $L_b = 10$ ft (3.05 m)].

The interaction equation, Equation H2-1, applies equally to the case where the axial force is in tension.

H3. MEMBERS UNDER TORSION AND COMBINED TORSION, FLEXURE, SHEAR AND/OR AXIAL FORCE

Section H3 provides provisions for cases not covered in the previous two sections. The first two parts of this section address the design of HSS members, and the third part is a general provision directed to cases where the designer encounters torsion in addition to normal stresses and shear stresses.

1. Torsional Strength of Round and Rectangular HSS

Hollow structural sections (HSS) are frequently used in space-frame construction and in other situations wherein significant torsional moments must be resisted by the members. Because of its closed cross section, an HSS is far more efficient in resisting torsion than an open cross section such as a W-shape or a channel. While normal and shear stresses due to restrained warping are usually significant in shapes of open cross section, they are insignificant in closed cross sections. The total torsional moment can be assumed to be resisted by pure torsional shear stresses. These are often referred in the literature as *St. Venant torsional* stresses.

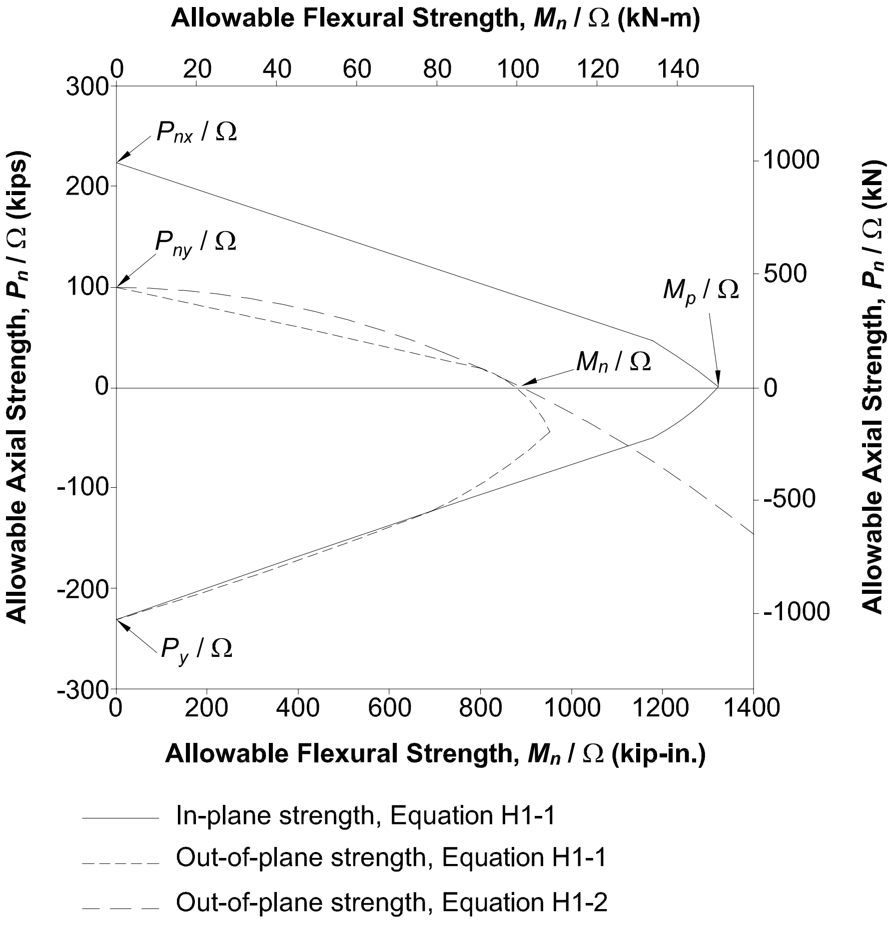


Fig. C-H1.6. Beam-columns under compressive and tensile axial force (tension is shown as negative) (ASD) [W16×26 (W410×38.8), $F_y = 50$ ksi (345 MPa), $L_b = 10$ ft (3.05 m)].

The pure torsional shear stress in HSS sections is assumed to be uniformly distributed along the wall of the cross section, and it is equal to the torsional moment, T_u, divided by a torsional shear constant for the cross section, C. In a limit state format, the nominal torsional resisting moment is the shear constant times the critical shear stress, F_{cr}.

For round HSS, the torsional shear constant is equal to the polar moment of inertia divided by the radius, which leads to

$$C = \frac{\pi t (D - t)^2}{2} \qquad \text{(C-H3-1)}$$

For rectangular HSS, the torsional shear constant is obtained as $2tA_o$ using the membrane analogy (Timoshenko, 1956), where A_o is the area bounded by the midline of the section. Conservatively assuming an outside corner radius of $2t$, the midline radius is $1.5t$ and

$$A_o = t^2 (B - t)(H - t) \frac{9(4 - \pi)}{4} \qquad \text{(C-H3-2)}$$

resulting in

$$C = 2t (B - t)(H - t) - 4.5t^3 (4 - \pi) \qquad \text{(C-H3-3)}$$

The resistance factor ϕ and the safety factor Ω are the same as for flexural shear in Chapter G.

When considering local buckling in round HSS subjected to torsion, most structural members will either be long or of moderate length and the provisions for short cylinders will not apply. The elastic local buckling strength of long cylinders is unaffected by end conditions and the critical stress is given in Galambos (1998) as

$$F_{cr} = \frac{K_t E}{\left(\frac{D}{t}\right)^{\frac{3}{2}}} \qquad \text{(C-H3-4)}$$

The theoretical value of K_t is 0.73 but a value of 0.6 is recommended to account for initial imperfections. An equation for the elastic local buckling stress for round HSS of moderate length ($L > 5.1D^2/t$) where the edges are not fixed at the ends against rotation is given in Schilling (1965) and Galambos (1998) as

$$F_{cr} = \frac{1.23E}{\left(\frac{D}{t}\right)^{\frac{5}{4}} \sqrt{\frac{L}{D}}} \qquad \text{(C-H3-5)}$$

This equation includes a 15 percent reduction to account for initial imperfections. The length effect is included in this equation for simple end conditions, and the approximately 10 percent increase in buckling strength is neglected for edges fixed at the end. A limitation is provided so that the shear yield strength $0.6F_y$ is not exceeded.

The critical stress provisions for rectangular HSS are identical to the flexural shear provisions of Section G2 with the shear buckling coefficient equal to $k_v = 5.0$. The shear distribution due to torsion is uniform in the longest sides of a rectangular HSS, and this is the same distribution that is assumed to exist in the web of a W-shape beam. Therefore, it is reasonable that the provisions for buckling are the same in both cases.

2. HSS Subject to Combined Torsion, Shear, Flexure and Axial Force

Several interaction equation forms have been proposed in the literature for load combinations that produce both normal and shear stresses. In one common form, the normal and shear stresses are combined elliptically with the sum of the squares (Felton and Dobbs, 1967):

$$\left(\frac{f}{F_{cr}}\right)^2 + \left(\frac{f_v}{F_{vcr}}\right)^2 \leq 1 \qquad \text{(C-H3-6)}$$

In a second form, the first power of the ratio of the normal stresses is used:

$$\left(\frac{f}{F_{cr}}\right) + \left(\frac{f_v}{F_{vcr}}\right)^2 \leq 1 \qquad \text{(C-H3-7)}$$

The latter form is somewhat more conservative, but not overly so (Schilling, 1965), and this is the form used in this Specification:

$$\left(\frac{P_r}{P_c} + \frac{M_r}{M_c}\right) + \left(\frac{V_r}{V_c} + \frac{T_r}{T_c}\right)^2 \leq 1.0 \qquad \text{(C-H3-8)}$$

where the terms with the subscript r represent the required strengths, and the ones with the subscript c are the corresponding available strengths. Normal effects due to flexural and axial load effects are combined linearly and then combined with the square of the linear combination of flexural and torsional shear effects. When an axial compressive load effect is present, the required flexural strength, M_c, is to be determined by second-order analysis.

3. Strength of Non-HSS Members under Torsion and Combined Stress

This section covers all the cases not previously covered. Examples are built-up unsymmetric crane-girders and many other types of odd-shaped built-up cross sections. The required stresses are determined by elastic stress analysis based on established theories of structural mechanics. The three limit states to consider and the corresponding available stresses are:

(1) Yielding under normal stress—F_y
(2) Yielding under shear stress—$0.6F_y$
(3) Buckling—F_{cr}

In most cases it is sufficient to consider normal stresses and shear stresses separately because maximum values rarely occur in the same place in the cross section or at the same place in the span. Seaburg and Carter (1997) provides a complete discussion on torsional analysis of open shapes.

CHAPTER I

DESIGN OF COMPOSITE MEMBERS

Chapter I includes extensive technical and format changes as well as significant new material when compared to previous editions of the Specification. The major technical changes consist of new design provisions for composite columns (Section I2), which now include new cross-sectional strength models, provisions for tension and shear design, and a liberalization of the slenderness limits for HSS. Other significant technical changes have been made in the shear stud strength provisions (Section I3.2d): the use of an ultimate strength model for ASD design of composite beams (Section I3.2) and new material limitations (Section I1.2).

The main format changes in Chapter I include the elimination of the former Section I1, Design Assumptions and Definitions. The contents of that Section have been moved to the Glossary, the notation section, or other locations in the Specification and the section has been replaced by a section on General Provisions. Other format changes are as follows: the separation of composite column design into distinct provisions for concrete-encased sections and concrete-filled sections; and the incorporation of the former Section I5, Shear Connectors, into the current Section I3. In addition, the extensive historical notes on the development of composite design provisions present in the Commentary of the previous editions of the Specification have been eliminated as that material is now considered to be widely known.

I1. GENERAL PROVISIONS

Design of composite sections requires consideration of both steel and concrete behavior. These provisions were developed with the intent both to minimize conflicts between current steel and concrete design provisions and to give proper recognition to the advantages of composite design. As a result of the attempt to minimize conflicts, this Specification now uses a cross-sectional strength approach for column design consistent with that used in reinforced concrete design (ACI, 2002). This approach, in addition, results in a consistent treatment of cross-sectional strengths for both composite columns and beams.

This Specification assumes that the user is familiar with reinforced concrete design specifications such as ACI (2002) and does not repeat many of the provisions needed for the concrete portion of the design, such as material specifications, anchorage and splice lengths, and shear and torsion provisions.

The provisions in Chapter I address strength design of the composite sections only. The designer needs to consider the loads resisted by the steel section alone when determining load effects during the construction phase. The designer also needs to consider deformations throughout the life of the structure and the appropriate cross section for those deformations. When considering these latter limit states, due allowance should be made for the additional long-term changes in stresses and deformations due to creep and shrinkage of the concrete.

1. Nominal Strength of Composite Sections

The strength of composite sections shall be computed based on either of the two approaches presented in this Specification. The first is the strain compatibility approach, which provides a general calculation method. The second is the plastic stress distribution approach, which is a subset of the strain compatibility approach. The plastic stress distribution method provides a simple and convenient calculation method for the most common design situations, and is thus treated first.

1a. Plastic Stress Distribution Method

The plastic stress distribution method is based on the assumption of linear strain across the cross section and elasto-plastic behavior. It assumes that the concrete has reached its crushing strength in compression at a strain of 0.003 and a corresponding stress (typically $0.85 f_c'$) on a rectangular stress block, and that the steel has exceeded its yield strain, typically taken as F_y/E_s.

Based on these simple assumptions, the cross-sectional strength for different combinations of axial force and bending moment may be approximated, for typical composite column cross-sections. The actual interaction diagram for moment and axial force for a composite section based on a plastic stress distribution is similar to that of a reinforced concrete section as shown in Figure C-I1.1. As a simplification, for concrete-encased sections, a conservative linear interaction between four or five anchor points, depending on axis of bending, can be used (Roik and Bergmann, 1992; Galambos, 1998). These points are identified as A, B, C, D and E in Figure C-I1.1.

The plastic stress approach for columns assumes that no slip has occurred between the steel and concrete portions and that the required width-to-thickness ratios prevent local buckling from occurring until extensive yielding has taken place. Tests and analyses have shown that these are reasonable assumptions at the ultimate limit states for both concrete-encased steel sections with shear connectors and for

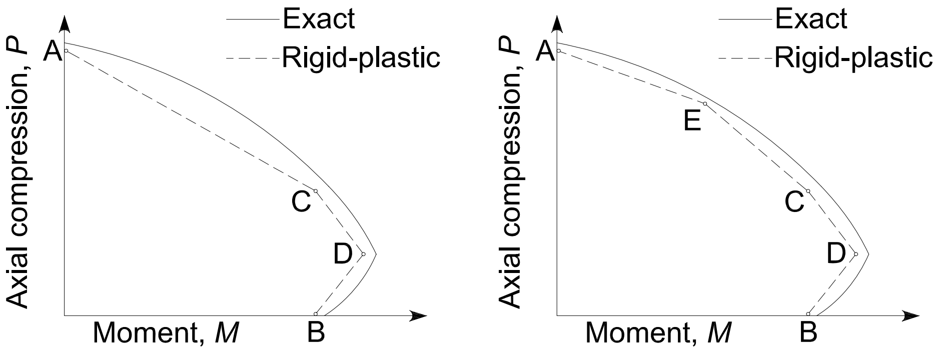


Fig. C-I1.1. Comparison between exact and simplified moment-axial compressive force envelopes.

HSS sections that comply with these provisions (Galambos, 1998; Hajjar, 2000; Shanmugam and Lakshmi, 2001). For circular HSS, these provisions allow for the increase of the usable concrete stress to $0.95f'_c$ to account for the beneficial effects of the restraining hoop action arising from transverse confinement (Leon and Aho, 2002).

Based on similar assumptions, but allowing for slip between the steel beam and the composite slab, simplified expressions can also be derived for typical composite beam sections. Strictly speaking, these distributions are not based on slip, but on the strength of the shear connection. Full interaction is assumed if the shear connection strength exceeds that of either (a) the tensile yield strength of the steel section or the compressive strength of the concrete slab when the composite beam is loaded in positive moment, or (b) the tensile yield strength of the longitudinal reinforcing bars in the slab or the compressive strength of the steel section when loaded in negative moment. When shear connectors are provided in sufficient numbers to fully develop this flexural strength, any slip that occurs prior to yielding has a negligible affect on behavior. When full interaction is not present, the beam is said to be partially composite. The effects of slip on the elastic properties of a partially composite beam can be significant and should be accounted for, if significant, in calculations of deflections and stresses at service loads. Approximate elastic properties of partially composite beams are given in Commentary Section I3.

1b. Strain-Compatibility Approach

The principles used to calculate cross-sectional strength in Section I1.1a may not be applicable to all design situations or possible cross-sections. As an alternative, Section I1.1b permits the use of a generalized strain-compatibility approach that allows the use of any reasonable strain-stress model for the steel and concrete.

2. Material Limitations

The material limitations given in Section I1.2 reflect the range of material properties available from experimental testing (Galambos, 1998; Hajjar, 2000; Shanmugam and Lakshmi, 2001; Leon and Aho, 2002). As for reinforced concrete design, a limit of 10 ksi (70 MPa) is imposed for strength calculations, both to reflect the scant data available above this strength and the changes in behavior observed, particularly for brittle failure modes such as shear. A lower limit of 3 ksi (21 MPa) is specified for both normal and lightweight concrete and an upper limit of 6 ksi (42 MPa) is specified for lightweight concrete to encourage the use of good quality, yet readily available, grades of structural concrete. The use of higher strengths in computing the modulus of elasticity is permitted, and the limits given can be extended for strength calculations if appropriate testing and analyses are carried out.

3. Shear Connectors

This section provides basic shear connector details and material specifications. Nominal yield and tensile strengths of typical ASTM A108 Type B studs are 51 ksi (350 MPa) and 65 ksi (450 MPa), respectively (AWS 2004).

I2. AXIAL MEMBERS

In Section I2, the design of concrete-encased and concrete-filled composite columns is treated separately, although they have much in common. The intent is to facilitate design by keeping the general principles and detailing requirements for each type of column separate.

An ultimate strength cross-section model is used to determine the section strength (Leon and Aho, 2002). This model is similar to that used in previous LRFD Specifications. The major difference is that the full strength of the reinforcing steel and concrete are accounted for rather than the 70 percent that was used in those previous specifications. In addition, these provisions give the strength of the composite section as a force, while the previous approach had converted that force to an equivalent stress. Since the reinforcing steel and concrete had been arbitrarily discounted, the previous provisions did not accurately predict strength for columns with a low percentage of steel.

The design for length effects is consistent with that for steel columns. The equations used are the same as those in Chapter E, albeit in a slightly different format, and as the percent of concrete in the section decreases, the design defaults to that of a steel section. Comparisons between the provisions in the Specification and experimental data show that the method is generally conservative but that the coefficient of variation obtained is large (Leon and Aho, 2002).

1. Encased Composite Columns

1a. Limitations

(1) In this Specification, the use of composite columns is extended from the previous minimum steel ratio of 4 percent (area of steel shape divided by the gross area of the member) down to columns with a minimum of 1 percent. This is a direct result of using an ultimate strength cross-sectional approach, and removes the previous discontinuities in design that occurred as the steel ratio decreased below 4 percent.

(2) The specified minimum quantity for transverse reinforcement is intended to provide good confinement to the concrete.

(3) A minimum amount of longitudinal reinforcing steel is prescribed so that at least four continuous corner bars are used (see Section I2.1f). Other longitudinal bars may be needed to provide the required restraint to the cross-ties, but that longitudinal steel cannot be counted towards the cross-sectional strength unless it is continuous and properly anchored. It is expected that the limit will seldom be reached in practice, except for the case of a very large cross section.

1b. Compressive Strength

The compressive strength of the cross section is given as the sum of the ultimate strengths of the components. The strength is not capped as in reinforced concrete column design for a combination of the following reasons: (1) the resistance factor has been lowered from 0.85 in previous editions to 0.75 in this Specification; (2) the required transverse steel provides better performance than a typical reinforced concrete column; (3) the presence of a steel section near the center of the section reduces the possibility of a sudden failure due to buckling of the longitudinal reinforcing steel; and (4) in most cases there will be significant load eccentricities (in other words, moments) present due to the size of the member and the typical force introduction mechanisms.

1c. Tensile Strength

The new Section I2.1c has been added to clarify the tensile strength to be used in situations where uplift is a concern and for computations related to beam-column interaction. The provision focuses on the limit state of yield on gross area. Where appropriate for the structural configuration, consideration should also be given to other tensile strength and connection strength limit states as specified in Chapters D and J.

1d. Shear Strength

This new material has been added to provide guidance for the shear strength of composite columns. The provisions require either the use of the steel section alone plus the contribution from any transverse shear reinforcement present in the form of ties or the shear strength calculated based on the reinforced concrete portion of the cross-section alone (in other words, longitudinal and transverse reinforcing bars plus concrete). This implies the following shear strengths:

$$V_n = 0.6F_y A_w + A_{st} F_{yr} \frac{d}{s}$$

$$\phi = 0.9 \text{ (LRFD)} \qquad \Omega = 1.67 \text{ (ASD)}$$

or

$$V_n = 2\sqrt{f'_c}\, bd + A_{st} F_{yr} \frac{d}{s}$$

$$\phi = 0.75 \text{ (LRFD)} \qquad \Omega = 2.00 \text{ (ASD)}$$

It would be logical to suggest provisions where both the contributions of the steel section and the reinforced concrete are superimposed; however, there is little research available on this topic.

1e. Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections in encased composite columns, a transfer of load by direct bearing, shear connection, or a combination of both is required. Although it is recognized that force transfer also occurs by direct bond interaction between the steel and concrete, this is typically ignored for encased composite columns (Griffis, 1992).

When shear connectors are used in encased composite columns, a uniform spacing is appropriate in most situations, but when large forces are applied, other connector arrangements may be needed to avoid overloading the component (steel section or concrete encasement) to which the load is applied directly.

When a supporting concrete area is wider on all sides than the loaded area, the nominal bearing strength for concrete may be taken as

$$N_b = 0.85 f_c' \sqrt{A_2/A_1} \tag{C-I2-1}$$

where A_1 is the loaded area and A_2 is the maximum area of the supporting surface that is geometrically similar and concentric with the loaded area. The value of $\sqrt{A_2/A_1}$ must be less than or equal to 2. This Specification uses the maximum nominal bearing strength of $1.7 f_c' A_B$. The resistance factor for bearing, ϕ_B, is 0.65 (and the associated safety factor Ω_B is 2.31) in accordance with ACI (2002).

2. Filled Composite Columns

2a. Limitations

(1) As discussed for encased columns, it is now permissible to design composite columns with a steel ratio as low as 1 percent.

(2) The specified minimum wall slenderness has been liberalized from previous editions of the *LRFD Specification*. Those editions did not differentiate between buckling of an unfilled and a filled HSS. The new provisions take into account the restraining effect of the concrete on the local buckling of the section wall.

2b. Compressive Strength

The compressive strength of the cross section is given as the sum of the ultimate strengths of the components. The beneficial confining effect of a circular HSS can be taken into account by increasing the crushing strength of the concrete to $0.95 f_c'$.

2c. Tensile Strength

As for encased columns, this new Section I2.2c has been added to clarify tensile strength.

2d. Shear Strength

See commentary to Section I2.1d.

2e. Load Transfer

To avoid overstressing either the structural steel section or the concrete at connections in filled composite columns, a transfer of load by direct bearing, shear connection, or direct bond interaction is permitted, with the mechanism providing the largest resistance being permissible for use. However, superposition of these force transfer mechanisms is not permitted for filled composite columns, as the experimental data indicate that direct bearing or shear connection often do not initiate until after direct bond interaction has been breached, and little experimental data is available about the interaction of direct bearing and shear interaction in filled composite columns.

Force transfer by direct bond is commonly used in filled composite columns as long as the connections are detailed to limit local deformations (API, 1993; Roeder and others, 1999). However, there is large scatter in the experimental data on the bond strength and associated force transfer length of filled composite columns, particularly when comparing tests in which the concrete core is pushed through the steel tube (push-out tests) to tests in which a beam is connected just to the steel tube and beam shear is transferred to the filled composite column. The added eccentricities of the connection tests typically raise the bond strength of the filled composite columns.

A reasonable lower bound value of bond strength of filled composite columns that meet the provisions of Section I2 is 60 psi (0.4 MPa). While push-out tests often show bond strengths below this value, eccentricity introduced into the connection is likely to increase the bond strength to this value or higher. Experiments also indicate that a reasonable assumption for the distance along the length of the filled composite column required to transfer the force from the steel HSS to the concrete core is approximately equal to the width of a rectangular HSS or the diameter of a round HSS, both above and below the point of load transfer.

One approach to estimating the direct bond interaction for filled HSS is presented below with recommendations for ϕ and Ω. These equations assume that one face of a rectangular filled composite column, or one-half of the perimeter of a circular filled composite column, is engaged in the transfer of stress by direct bond interaction. Higher values of nominal bond strength may be warranted for specific conditions. The scatter in the data leads to the recommended low value of the resistance factor, ϕ, and the corresponding high value of the safety factor, Ω.

(a) For rectangular HSS filled with concrete:

$$V_{in} = b^2 C_{in} F_{in} \tag{C-I2-2}$$

$$\phi = 0.45 \text{ (LRFD)} \qquad \Omega = 3.33 \text{ (ASD)}$$

where

V_{in} = nominal bond strength, kips (N)
F_{in} = nominal bond stress = 60 psi (0.40 MPa)
b = width of HSS along face transferring load, in. (mm)
C_{in} = 1 if the filled composite column extends only above or below the point of load transfer
= 2 if the filled composite column extends both above and below the point of load transfer

(b) For round HSS filled with concrete:

$$V_{in} = 0.5\pi D^2 C_{in} F_{in} \tag{C-I2-3}$$

$$\phi = 0.45 \text{ (LRFD)} \qquad \Omega = 3.33 \text{ (ASD)}$$

where

V_{in} = nominal bond strength, kips (N)
F_{in} = nominal bond stress = 60 psi (0.40 MPa)
D = diameter of HSS, in. (mm)

C_{in} = 1 if the filled composite column extends only above or below the point of load transfer
= 2 if the filled composite column extends both above and below the point of load transfer

As with encased columns, this specification assumes that the most advantageous combination of loaded area and concrete area are used to determine bearing strength. Thus, the nominal bearing strength is taken as $1.7 f_c' A_B$.

2f. Detailing Requirements

When shear connectors are used in filled composite columns, the provisions require that they be placed a distance of 2.5 times the width of a rectangular HSS or 2.5 times the diameter of a round HSS, both above and below the load transfer region. In most such situations, a uniform spacing is appropriate. However, when large forces are applied, other connector arrangements may be needed to avoid overloading the steel section or concrete core to which the load is applied directly.

I3. FLEXURAL MEMBERS

1. General

Three types of composite beams are addressed in this section: fully encased steel beams, concrete-filled HSS, and steel beams with mechanical anchorage to the slab.

When a composite beam is controlled by deflection, the design should limit the behavior of the beam to the elastic range under serviceability load combinations. Alternatively, the amplification effects of inelastic behavior should be considered when deflection is checked.

It is often not practical to make accurate stiffness calculations of composite flexural members. Comparisons to short-term deflection tests indicate that the effective moment of inertia, I_{eff}, is 15 to 30 percent lower than that calculated based on linear elastic theory (I_{equiv}). Therefore, for realistic deflection calculations, I_{eff} should be taken as 0.75 I_{equiv}.

As an alternative, one may use a lower bound moment of inertia, I_{lb}, as defined below:

$$I_{lb} = I_s + A_s(Y_{ENA} - d_3)^2 + (\Sigma Q_n / F_y)(2d_3 + d_1 - Y_{ENA})^2 \qquad \text{(C-I3-1)}$$

where

A_s = area of steel cross section, in.^{2} (mm^2)
d_1 = distance from the compression force in the concrete to the top of the steel section, in. (mm)
d_3 = distance from the resultant steel tension force for full section tension yield to the top of the steel, in. (mm)
I_{lb} = lower bound moment of inertia, in.^{4} (mm^4)

I_s = moment of inertia for the structural steel section, in.^{4} (mm^4)

ΣQ_n = sum of the nominal strengths of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kips (kN)

$Y_{ENA} = [(A_s d_3 + (\Sigma Q_n / F_y)(2d_3 + d_1))/(A_s + (\Sigma Q_n / F_y))]$

The use of constant stiffness in elastic analyses of continuous beams is analogous to the practice in reinforced concrete design. The stiffness calculated using a weighted average of moments of inertia in the positive moment region and negative moment regions may take the following form:

$$I_t = aI_{pos} + bI_{neg} \qquad \text{(C-I3-2)}$$

where

I_{pos} = effective moment of inertia for positive moment, in.^{4} (mm^4)

I_{neg} = effective moment of inertia for negative moment, in.^{4} (mm^4)

The effective moment of inertia is based on the cracked transformed section considering the degree of composite action. For continuous beams subjected to gravity loads only, the value of *a* may be taken as 0.6 and the value of *b* may be taken as 0.4. For composite beams used as part of a lateral force resisting system in moment frames, the value of *a* and *b* may be taken as 0.5 for calculations related to drift.

In cases where elastic behavior is desired, the cross-sectional strength of composite members is based on the superposition of elastic stresses including consideration of the effective section modulus at the time each increment of load is applied. For cases where elastic properties of partially composite beams are needed, the elastic moment of intertia may be approximated by

$$I_{eff} = I_s + \sqrt{(\Sigma Q_n / C_f)}\,(I_{tr} - I_s) \qquad \text{(C-I3-3)}$$

where

I_s = moment of inertia for the structural steel section, in.^{4} (mm^4)

I_{tr} = moment of inertia for the fully composite uncracked transformed section, in.^{4} (mm^4)

ΣQ_n = strength of shear connectors between the point of maximum positive moment and the point of zero moment to either side, kips (N)

C_f = compression force in concrete slab for fully composite beam; smaller of $A_s F_y$ and $0.85 f'_c A_c$, kips (N)

A_c = area of concrete slab within the effective width, in.^{2} (mm^2)

The effective section modulus S_{eff}, referred to the tension flange of the steel section for a partially composite beam, may be approximated by

$$S_{eff} = S_s + \sqrt{(\Sigma Q_n / C_f)}\,(S_{tr} - S_s) \qquad \text{(C-I3-4)}$$

where

S_s = section modulus for the structural steel section, referred to the tension flange, in.^{3} (mm^3)
S_{tr} = section modulus for the fully composite uncracked transformed section, referred to the tension flange of the steel section, in.^{3} (mm^3)

Equations C-I3-3 and C-I3-4 should not be used for ratios, $\Sigma Q_n / C_f$, less than 0.25. This restriction is to prevent excessive slip, as well as substantial loss in beam stiffness. Studies indicate that Equations C-I3-3 and C-I3-4 adequately reflect the reduction in beam stiffness and strength, respectively, when fewer connectors are used than required for full composite action (Grant and others, 1977).

U.S. practice does not generally require the following items be considered. They are highlighted here for a designer who chooses to construct something for which these items might apply.

1. Horizontal shear strength of the slab: For the case of girders with decks with narrow troughs or thin slabs, shear strength of the slab may govern the design (for example, see Figure C-I3.1). Although the configuration of decks built in the U.S. tends to preclude this mode of failure, it is important that it be checked if the force in the slab is large or an unconventional assembly is chosen. The shear strength of the slab may be calculated as the superposition of the shear strength of the concrete plus the contribution of any slab steel crossing the shear plane. The required shear strength, as shown in the figure, is given by the difference in the force between the regions inside and outside the potential failure surface. Where experience has shown that longitudinal cracking detrimental to serviceability is likely to occur, the slab should be reinforced in the direction transverse to the supporting steel section. It is recommended that the area of such reinforcement be at least 0.002 times the concrete area in the longitudinal direction of the beam and that it be uniformly distributed.

2. Rotational capacity of hinging zones: There is no required rotational capacity for hinging zones. Where plastic redistribution to collapse is allowed, the moments

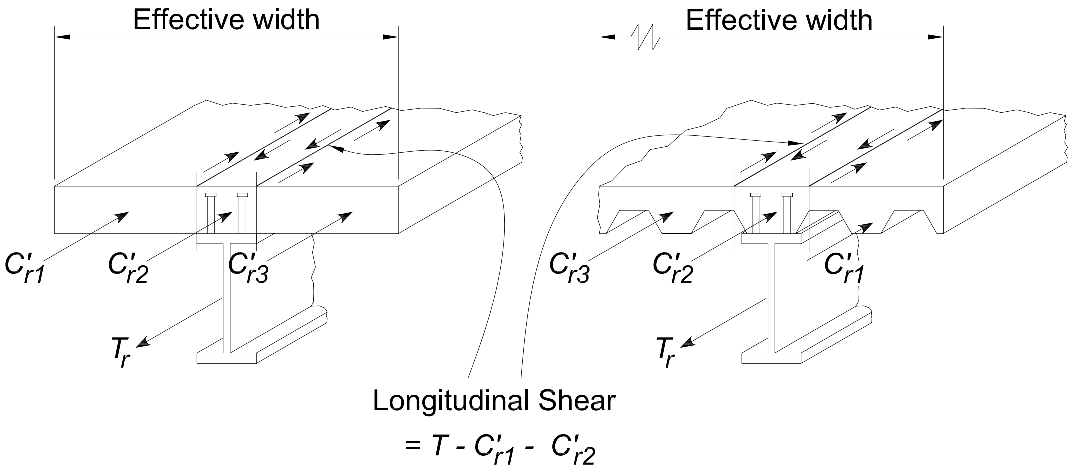


Fig. C-I3.1. Longitudinal shear in the slab (after Chien and Ritchie, 1984).

at a cross section may be as much as 30 percent lower than those given by a corresponding elastic analysis. This reduction in load effects is predicated, however, on the ability of the system to deform through very large rotations. To achieve these rotations, very strict local buckling and lateral-torsional buckling requirements must be fulfilled (Dekker and others, 1995). For cases in which a 10 percent redistribution is utilized (see Appendix 1), the required rotation capacity is within the limits provided by the local and lateral-torsional buckling provisions of Chapter F. Therefore, a rotational capacity check is not normally required for designs using this provision.

3. Minimum amount of shear connection: There is no minimum requirement for the amount of shear connection. Design aids in the U.S. often limit partial composite action to a minimum of 25 percent for practical reasons, but two issues arise with the use of low degrees of partial composite action. First, less than 50 percent composite action requires large rotations to reach the available flexural strength of the member and can result in very limited ductility after the nominal strength is reached. Second, low composite action results in an early departure from elastic behavior in both the beam and the studs. The current provisions, which are based on ultimate strength concepts, have eliminated checks for ensuring elastic behavior under service load combinations, and this can be an issue if low degrees of partial composite action are used.

4. Long-term deformations due to shrinkage and creep: There is no direct guidance in the computation of the long-term deformations of composite beams due to creep and shrinkage. The long-term deformation due to shrinkage can be calculated with the simplified model shown in Figure C-I3.2, in which the effect of shrinkage is taken as an equivalent set of end moments given by the shrinkage force (long-term restrained shrinkage strain times modulus of concrete times effective area of concrete) times the eccentricity between the center of the slab and the elastic neutral axis. If the restrained shrinkage coefficient for the aggregates is not known, the shrinkage strain for these calculations may be taken as 0.02 percent. The long-term deformations due to creep, which can be quantified using a model similar to that shown in the figure, are small unless the spans are long and the permanent live loads large. For shrinkage and creep effects, special attention should be given to lightweight aggregates, which tend to have higher creep coefficients and moisture absorption and lower modulus of elasticity than conventional aggregates, exacerbating any potential deflection problems. Engineering judgment is required, as calculations for long-term deformations require consideration of the many variables involved and because linear superposition of these effects is not strictly correct (ACI, 1997; Viest and others, 1997).

1a. Effective Width

The same effective width rules apply to composite beams with a slab on either one side or both sides of the beam. In cases where the effective stiffness of a beam with a one-sided slab is important, special care should be exercised since this model can substantially overestimate stiffness (Brosnan and Uang, 1995). To simplify design, effective width is based on the full span, center-to-center of supports, for both simple and continuous beams.

1b. Shear Strength

A conservative approach to shear provisions for composite beams is adopted by assigning all shear to the steel section web. This neglects any concrete slab contribution and serves to simplify design.

1c. Strength during Construction

Composite beam design requires care in considering the loading history. Loads applied to an unshored beam before the concrete has cured are resisted by the steel section alone, and only loads applied after the concrete has cured are considered to be resisted by the composite section. It is usually assumed for design purposes that concrete has hardened when it attains 75 percent of its design strength. Unshored beam deflection caused by fresh concrete tends to increase slab thickness and dead load. For longer spans this may lead to instability analogous to roof ponding. Excessive increase of slab thickness may be avoided by beam camber. Pouring the slab to a constant thickness will also help eliminate the possibility of ponding instability (Ruddy, 1986). When forms are not attached to the top flange, lateral bracing of the steel beam during construction may not be continuous and the unbraced length may control flexural strength, as defined in Chapter F.

This Specification does not include special requirements for strength during construction. For these noncomposite beams, the provisions of Chapter F apply.

Load combinations for construction loads should be determined for individual projects according to local conditions, using ASCE (2002) as a guide.

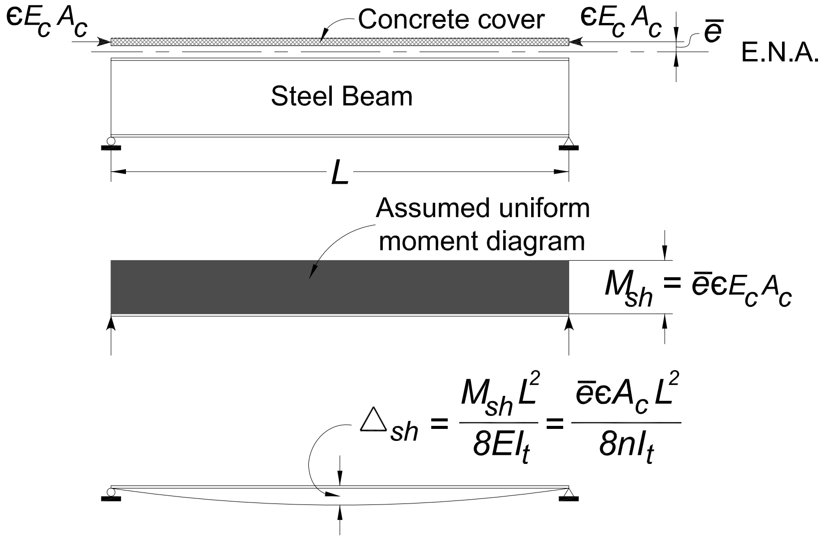


Fig. C-I3.2. Calculation of shrinkage effects [from Chien and Ritchie (1984)].

2. Strength of Composite Beams with Shear Connectors

Section I3.2 applies to simple and continuous composite beams with shear connectors, constructed with or without temporary shores.

2a. Positive Flexural Strength

The flexural strength of a composite beam in the positive moment region may be controlled by the strength of the steel section, the concrete slab or the shear connectors. In addition, web buckling may limit flexural strength if the web is slender and a large portion of the web is in compression.

According to Table B5.1, local web buckling does not reduce the plastic strength of a bare steel beam if the beam depth-to-web thickness ratio is not larger than $3.76\sqrt{E/F_y}$. In the absence of web buckling research on composite beams, the same ratio is conservatively applied to composite beams.

For beams with more slender webs, this Specification conservatively adopts first yield as the flexural strength limit. In this case, stresses on the steel section from permanent loads applied to unshored beams before the concrete has cured must be superimposed on stresses on the composite section from loads applied to the beams after hardening of concrete. For shored beams, all loads may be assumed to be resisted by the composite section.

When first yield is the flexural strength limit, the elastic transformed section is used to calculate stresses on the composite section. The modular ratio, $n = E/E_c$, used to determine the transformed section, depends on the specified unit weight and strength of concrete.

2b. Negative Flexural Strength

Loads applied to a continuous composite beam with shear connectors throughout its length, after the slab is cracked in the negative moment region, are resisted in that region by the steel section and by properly anchored longitudinal slab reinforcement. When an adequately braced compact steel section and adequately developed longitudinal reinforcing bars act compositely in the negative moment region, the nominal flexural strength is determined from plastic stress distributions.

2c. Strength of Composite Beams with Formed Steel Deck

Figure C-I3.3 is a graphic presentation of the terminology used in Section I3.2c.

The design rules for composite construction with formed steel deck are based upon a study (Grant and others, 1977) of the then-available test results. The limiting parameters listed in Section I3.2c were established to keep composite construction with formed steel deck within the available research data.

The minimum spacing of 18 in. for connecting composite decking to the support is intended to address a minimum uplift requirement during the construction phase prior to placing concrete.

2d. Shear Connectors

(1) Load Transfer for Positive Moment

When studs are used on beams with formed steel deck, they may be welded directly through the deck or through prepunched or cut-in-place holes in the deck. The usual procedure is to install studs by welding directly through the deck; however, when the deck thickness is greater than 16 gage (1.5 mm) for single thickness, or 18 gage (1.2 mm) for each sheet of double thickness, or when the total thickness of galvanized coating is greater than 1.25 ounces/sq. ft (0.38 kg/m^2), special precautions and procedures recommended by the stud manufacturer should be followed.

Composite beam tests in which the longitudinal spacing of shear connectors was varied according to the intensity of the static shear, and duplicate beams

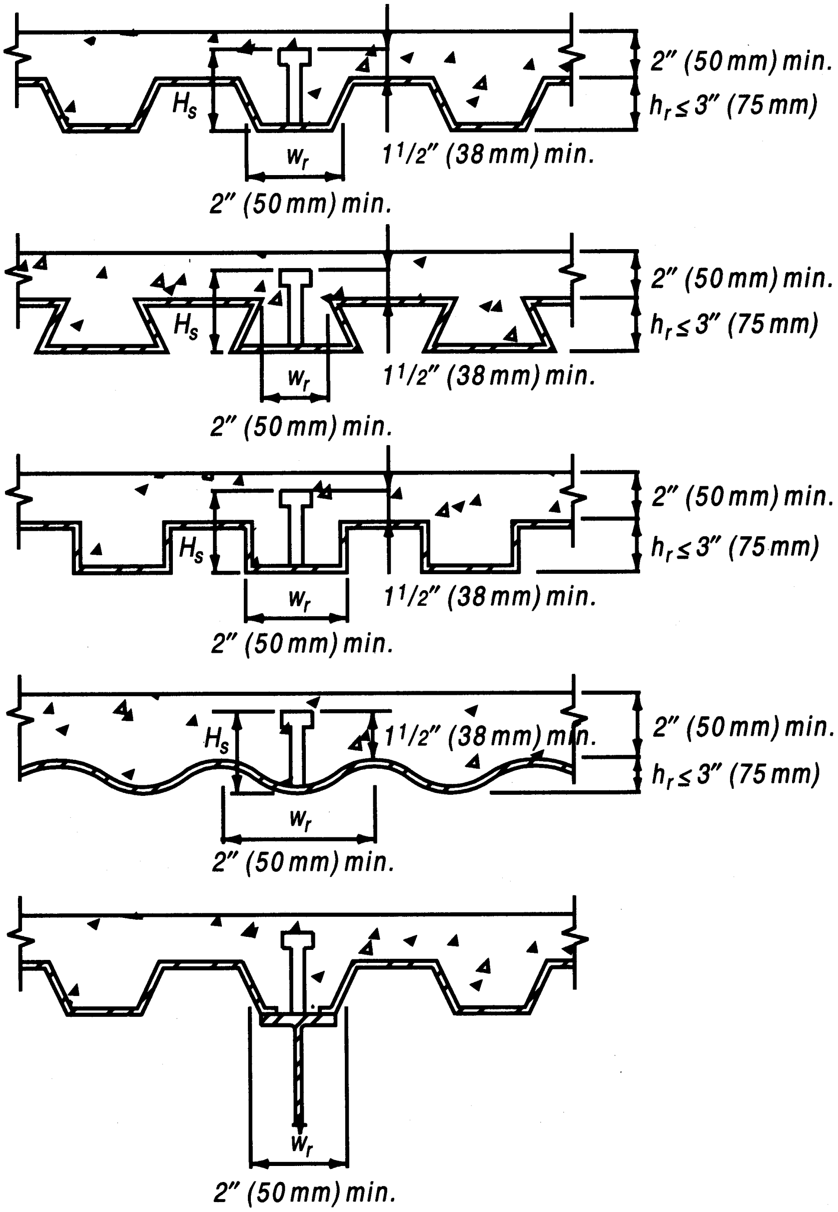


Fig. C-I3.3. Steel deck limits.

in which the connectors were uniformly spaced, exhibited approximately the same ultimate strength and approximately the same amount of deflection at nominal loads. Under distributed load conditions, only a slight deformation in the concrete near the more heavily stressed connectors is needed to redistribute the horizontal shear to other less heavily stressed connectors. The important consideration is that the total number of connectors be sufficient to develop the shear on either side of the point of maximum moment. The provisions of this Specification are based upon this concept of composite action.

In computing the available flexural strength at points of maximum negative bending, reinforcement parallel to the steel beam within the effective width of the slab may be included, provided such reinforcement is properly anchored beyond the region of negative moment. However, shear connectors are required to transfer the ultimate tensile force in the reinforcement from the slab to the steel beam.

When steel deck includes units for carrying electrical wiring, crossover headers are commonly installed over the cellular deck perpendicular to the ribs. These create trenches that completely or partially replace sections of the concrete slab above the deck. These trenches, running parallel to or transverse to a composite beam, may reduce the effectiveness of the concrete flange. Without special provisions to replace the concrete displaced by the trench, the trench should be considered as a complete structural discontinuity in the concrete flange.

When trenches are parallel to the composite beam, the effective flange width should be determined from the known position of the trench.

Trenches oriented transverse to composite beams should, if possible, be located in areas of low bending moment and the full required number of studs should be placed between the trench and the point of maximum positive moment. Where the trench cannot be located in an area of low moment, the beam should be designed as noncomposite.

(3) Strength of Stud Shear Connectors

Considerable research has been published in recent years indicating that shear stud strength equations in previous AISC Specifications are unconservative. Specifically, it has been understood for some time that the stud strength values given by Equation I5-1 in previous LRFD Specifications, in combination with the old Equations I3-2 and I3-3, which modified the strength based on whether the deck was perpendicular or parallel to the beams, are higher than those derived from either pushout or beam tests for studs embedded in modern steel decks (Jayas and Hosain, 1988; 1988a; Mottram and Johnson, 1990; Easterling, Gibbings, and Murray, 1993; Roddenberry and others, 2002) Equation I5-1 in the previous specifications is similar to the new Equation I3-5 but without the R_g and R_p factors.

Other codes use a stud strength expression similar to the previous AISC *LRFD Specification*; the stud strength is reduced by a ϕ factor of 0.8 in the Canadian code (CSA, 1994) and by an even lower partial safety factor ($\phi = 0.60$) for the corresponding stud strength equations in Eurocode 4 (2003).

The origin of this discrepancy can be traced to the way the old equations for stud strength were developed. The old approach was developed based on tests on solid slabs, and, as noted by the current R_p and R_g factors in the new Equation I3-4, the current approach remains valid for this case. Following studies reported in Robinson (1967) and Fisher (1970), Grant and others developed expressions for stud strength that accounted for the presence of the steel deck by including additional variables related to the deck and stud geometries (Grant and others, 1977). However, most of those tests were conducted with decks that were formed specifically for the tests from flat steel sheets.

The majority of composite steel floor decks used today have a stiffening rib in the middle of each deck flute. Because of the stiffener, studs must be welded off-center in the deck rib. Recent studies have shown that shear studs behave differently depending upon their location within the deck rib (Lawson, 1992; Easterling and others, 1993; Van der Sanden, 1995; Yuan, 1996; Johnson and Yuan, 1998; Roddenberry and others, 2002; Roddenberry and others, 2002a). The so-called “weak” (unfavorable) and “strong” (favorable) positions are illustrated in Figure C-I3.4. Furthermore, the maximum value shown in these studies for studs welded through steel deck is on the order of 0.7 to 0.75 $F_u A_{sc}$. Studs placed in the weak position have strengths as low as $0.5 F_u A_{sc}$.

The strength of stud connectors installed in the ribs of concrete slabs on formed steel deck with the ribs oriented perpendicular to the steel beam is reasonably estimated by the strength of stud connectors computed from Equation I3-3, which sets the default value for shear stud strength equal to that for the weak stud position. Both AISC (1997) and the Steel Deck Institute (SDI, 1999) recommend that studs be detailed in the strong position, but ensuring that studs are placed in the strong position is not necessarily an easy task because it is not always easy for the installer to determine where along the beam the particular rib is located, relative to the end, midspan or point of zero shear. Therefore, the installer may not be clear on which is the strong and which is the weak position.

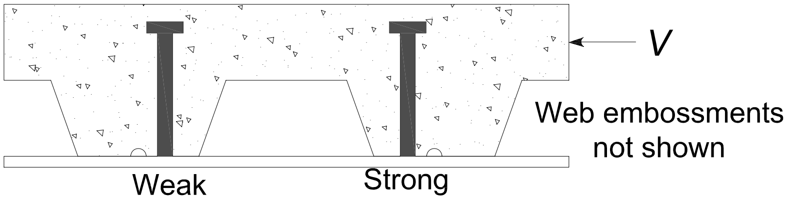


Fig. C-I3.4. Weak and strong stud positions [from Roddenberry and others (2002a)].

In most composite floors designed today, the ultimate strength of the composite section is governed by the stud strength, as full composite action is typically not the most economical solution to resist the required strength. The degree of composite action, as represented by the ratio $\Sigma Q_n / F_y A_s$ (the total shear connection strength divided by the yield strength of the steel cross section), influences the flexural strength as shown in Figure C-I3.5.

It can be seen from Figure C-I3.5 that a relatively large change in shear connection strength results in a much smaller change in flexural strength. Thus, formulating the influence of steel deck on shear connector strength by conducting beam tests and back-calculating through the flexural model, as was done in the past, lead to an inaccurate assessment of stud strength when installed in metal deck.

The changes in the 2005 Specification are not a result of either structural failures or performance problems. Designers concerned about the strength of existing structures need to note that the slope of the curve shown in Figure C-I3.5 is rather flat as the degree of composite action approaches one. Thus, even a large change in shear stud strength does not result in a proportional decrease of the flexural strength. In addition, as noted above, the current expression does not account for all the possible shear force transfer mechanisms, primarily because many of them are difficult or impossible to quantify. However, as noted in the Commentary to Section I3.1, as the degree of composite action decreases, the deformation demands on shear studs increase. This effect is reflected by the increasing slope of the relationship shown in Figure C-I3.5 as the degree of composite action decreases. Thus designers should be careful

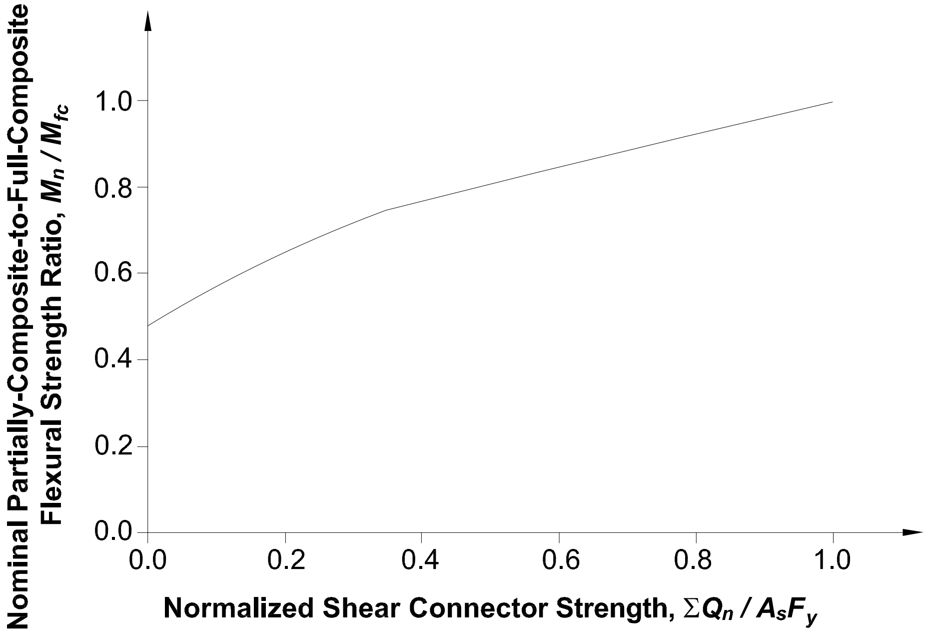


Fig. C-I3.5. Normalized flexural strength versus shear connection strength ratio [W16×31 (W410×46.1), $F_y = 50$ ksi (345 MPa), $Y2 = 4.5$ in. (114 mm)] (after Easterling and others, 1993).

when evaluating the strength of existing composite beams with 50 percent composite action or less.

(4) Strength of Channel Shear Connectors

Equation I3-4 is a modified form of the formula for the strength of channel connectors presented in Slutter and Driscoll (1965), which was based on the results of pushout tests and a few simply supported beam tests with solid slabs by Viest and others (1952). The modification has extended its use to lightweight concrete.

Eccentricities need not be considered in the weld design for cases where the welds at the toe and heel of the channel are greater than 3/16 in. and the connector meets the following requirements:

$$1.0 \le \frac{t_f}{t_w} \le 5.5$$

$$\frac{H}{t_w} \ge 8.0$$

$$\frac{L_c}{t_f} \ge 6.0$$

$$0.5 \le \frac{R}{t_w} \le 1.6$$

where t_f is the connector flange thickness, t_w is the connector web thickness, H is the height of the connector, L_c is the length of the connector, and R is the radius of the fillet between the flange and web of the connector.

(6) Shear Connector Placement and Spacing

Uniform spacing of shear connectors is permitted, except in the presence of heavy concentrated loads.

Studs not located directly over the web of a beam tend to tear out of a thin flange before attaining full shear-resisting strength. To guard against this contingency, the size of a stud not located over the beam web is limited to 2½ times the flange thickness (Goble, 1968). The practical application of this limitation is to select only beams with flanges thicker than the stud diameter divided by 2.5.

The minimum spacing of connectors along the length of the beam, in both flat soffit concrete slabs and in formed steel deck with ribs parallel to the beam, is six diameters; this spacing reflects development of shear planes in the concrete slab (Ollgaard and others, 1971). Because most test data are based on the minimum transverse spacing of four diameters, this transverse spacing was set as the minimum permitted. If the steel beam flange is narrow, this spacing requirement may be achieved by staggering the studs with a minimum transverse spacing of three diameters between the staggered row of studs. When deck ribs are parallel to the beam and the design requires more studs than can be placed in the rib, the deck may be split so that adequate spacing is available for stud installation. Figure C-I3.6 shows possible connector arrangements.

3. Flexural Strength of Concrete-Encased and Filled Members

Tests of concrete-encased beams demonstrated that: (1) the encasement drastically reduces the possibility of lateral-torsional instability and prevents local buckling of the encased steel; (2) the restrictions imposed on the encasement practically prevent bond failure prior to first yielding of the steel section; and (3) bond failure does not necessarily limit the moment strength of an encased steel beam (ASCE, 1979). Accordingly, this Specification permits three alternative design methods for determination of the nominal flexural strength: (a) based on the first yield in the tension flange of the composite section; (b) based on the plastic flexural strength of the steel section alone; and (c) based on the plastic flexural strength of the composite section or the strain-compatibility method. Method (c) is applicable only when shear connectors are provided along the steel section and reinforcement of the concrete encasement meets the specified detailing requirements. No limitations are placed on the slenderness of either the composite beam or the elements of the steel section, since the encasement effectively inhibits both local and lateral buckling.

In method (a), stresses on the steel section from permanent loads applied to unshored beams before the concrete has hardened must be superimposed on stresses on the composite section from loads applied to the beams after hardening of the concrete. In this superposition, all permanent loads should be multiplied by the dead load factor and all live loads should be multiplied by the live load factor. For shored beams, all loads may be assumed as resisted by the composite section. Complete interaction (no slip) between the concrete and steel is assumed.

I4. COMBINED AXIAL FORCE AND FLEXURE

As with all frame analyses in this Specification, required strengths for composite beam-columns should be obtained from second-order analysis or amplified first-order analysis. With respect to the assessment of the available strength, the Specification provisions for interaction between axial force and flexure in composite

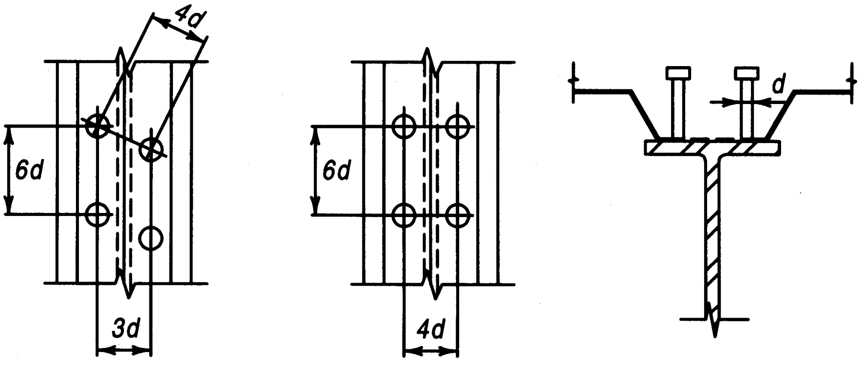


Fig. C-I3.6. Shear connector arrangements.

members do not provide explicit equations. However, the provisions provide guidance in Section I4 on the principles that can serve to establish an interaction diagram similar to those used in reinforced concrete design.

Composite Beam-Columns. The available axial strength, including the effects of buckling, and the available flexural strength can be calculated using either the plastic stress distribution method or the strain-compatibility method. There are three simplified approaches for determining the strength of composite beam-columns discussed below that take advantage of strength determination for a limited number of cases and interpolation for all other cases.

The first approach applies to doubly symmetric composite beam-columns, the most common geometry found in building construction. For this case, the interaction equations of Section H1 provide a conservative assessment of the available strength of the member for combined axial compression and flexure. These provisions may also be used for combined axial tension and flexure. The degree of conservatism generally depends on the extent of concrete contribution to the overall strength, relative to the steel contribution. Thus, for example, the equations are generally more conservative for members with high concrete compressive strength as compared to members with low concrete compressive strength.

The second approach is based on developing interaction surfaces for combined axial compression and flexure at the nominal strength level, using the plastic stress distribution method. This results in interaction surfaces similar to those shown in Figures C-I1.1 and C-I4.1. The five points identified in Figure C-I4.1 are defined by the plastic stress distribution used in their determination. Point *A* is the pure axial strength determined according to Equations I2-2 or I2-13. Point *B* is determined as the flexural strength of the section determined according to the provisions of Section I3. Point *C* corresponds to a plastic neutral axis location that results in the same flexural capacity as Point *B* but with axial load. Point *D* corresponds to an axial strength of one half of that determined for Point *C*. Point *E* is an arbitrary point necessary to better reflect bending strength for *y*-axis bending of encased shapes and bending of filled HSS. Linear interpolation between these anchor points may be used. However, with this approach, care should be taken in reducing Point *D* by a resistance factor or to account for member slenderness, as that may lead to an unsafe situation whereby additional flexural strength is permitted at a lower axial compressive strength than predicted by the cross section strength of the member. Once the nominal strength interaction surface is determined, length effects according to Equations I2-6 and I2-7 must be applied. The available strength is then determined by applying the compression and bending resistance factors or safety factors.

The third approach is a simplified bilinear approach as shown in Figure C-I4.1. After the column axial strength (Point *A* in Figure C-I4.1) is computed using Equation I2-2 for concrete-encased sections or Equation I2-13 for concrete-filled sections, this strength is reduced by the length effects using Equations E2-6 or E2-7 to obtain P_n, or Point A_λ. The resistance factor, ϕ_c, or safety factor, Ω_c, is then applied to this value to become the anchor point for design on the vertical axis, A_d. The anchor point on the horizontal axis, Point B_d, is given by the flexural strength of the section, Point B, modified by the appropriate bending resistance factor or safety factor.

Point C is then adjusted downward by the same length effect reduction as applied to Point A, to obtain Point C_λ. Point C_λ is then adjusted down by ϕ_c or Ω_c and to the left by ϕ_b or Ω_b to obtain Point C_d. A straight line approximation may then be used between Points A_d, C_d and B_d, as shown in the figure. Using linear interpolation between Points A_d, C_d and B_d in Figure C-I4.1, the following interaction equations may be derived for composite beam-columns subjected to combined axial compression plus biaxial flexure:

If $P_r < P_C$

$$\frac{M_{rx}}{M_{Cx}} + \frac{M_{ry}}{M_{Cy}} \leq 1 \tag{C-I4-1a}$$

If $P_r \geq P_C$

$$\frac{P_r - P_C}{P_A - P_C} + \frac{M_{rx}}{M_{Cx}} + \frac{M_{ry}}{M_{Cy}} \leq 1 \tag{C-I4-1b}$$

where

P_r = required compressive strength, kips (N)
P_A = available axial compressive strength at Point A, kips (N)
P_C = available axial compressive strength at Point C, kips (N)
M_r = required flexural strength, kip-in. (N-mm)
M_C = available flexural strength at Point C, kip-in. (N-mm)
x = subscript relating symbol to strong axis bending
y = subscript relating symbol to weak axis bending

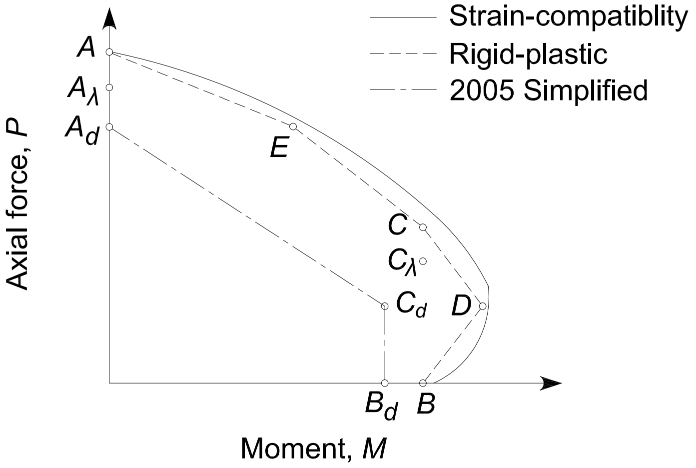


Fig. C-I4.1. Interaction diagram for composite beam-column design.

For design according to Section B3.3, (LRFD)

P_r $= P_u =$ required compressive strength using LRFD load combinations, kips (N)

P_A $= P_{Ad} =$ design axial compressive strength, determined in accordance with Section I2, kips (N)

P_C $= P_{Cd} =$ design axial compressive strength at point C_d in Figure C-I4.1, kips (N)

M_r $=$ required flexural strength using LRFD load combinations, kip-in. (N-mm)

$M_C = M_{Cd} = M_{Bd}$ design flexural strength, determined in accordance with Section I3, kip-in. (N-mm)

ϕ_c $= 0.75$

ϕ_b $= 0.90$

For design according to Section B3.4, (ASD)

P_r $= P_a =$ required compressive strength using ASD load combinations, kips (N)

P_A $= P_{Ad} =$ allowable compressive strength, determined in accordance with Section I2, kips (N)

P_C $= P_{Cd} =$ allowable axial compressive strength at Point C_d in Figure C-I4.1, kips (N)

M_r $=$ required flexural strength using ASD load combinations, kip-in. (N-mm)

$M_C = M_{Cd} = M_{Bd}$ allowable flexural strength, determined in accordance with Section I3, kip-in. (N-mm).

Ω_c $= 2.00$

Ω_b $= 1.67$

For biaxial bending, the value of P_C may be different when computed for the major and minor axis. The smaller of the two values should be used in Equation C-I4-1b and for the limits of Equations C-I4-1a and b.

Composite Beams Subjected to Combined Axial Force and Flexure. Combined axial force and flexure on composite floor beams has not been addressed directly in this chapter. Composite floor beam members (steel beams composite with floor slabs at their top flange) with axial loading may include collector elements (drag struts) and stabilizing elements for sloping column members. Few detailed design guidelines exist for such members; preliminary guidance for seismic design is given in AISC (2002).

Load combinations as set forth in ASCE (2002) should be used to determine the applicable loading at critical sections, including secondary bending effects of eccentrically applied loading. Adequate means to transmit axial loading to and from the steel section should be provided. Where shear connectors are used, the top flanges may be considered braced for compressive loading at the shear connector locations. Additional shear connectors may be required for axial load transfer and added flexure as determined from the load combinations. For shear studs added to transfer axial loads between beams and slabs, Q_n may be determined in accordance with Section I3. For load combinations resulting in compressive loading of the lower flange, length effects between brace points should be considered. Inflection points should not be considered brace points for torsional buckling of the unbraced flange. For discussion and design methodology, the reader is referred to Galambos (1998).

I5. SPECIAL CASES

Tests are required for construction that falls outside the limits given in this Specification. Different types of shear connectors may require different spacing and other detailing than stud and channel connectors.

CHAPTER J

DESIGN OF CONNECTIONS

The provisions of Chapter J cover the design of connections not subject to *cyclic loads*. Wind and other environmental loads are generally not considered to be *cyclic loads*. The provisions generally apply to connections other than HSS and box members. See Chapter K for HSS and box member connections and Appendix 3 for fatigue provisions.

J1. GENERAL PROVISIONS

Selection of weld type (CJP versus fillet versus PJP) depends on base connection geometry (butt versus T or corner) and required strength, among other issues discussed in this Section. Consideration of notch effects and the ability to evaluate with NDE may be appropriate for cyclically loaded joints or joints expected to deform plastically.

1. Design Basis

In the absence of defined design loads, a minimum design load should be considered. Historically, a value of 10 kips (44 kN) for LRFD and 6 kips (27 kN) for ASD have been used as reasonable values. For elements such as lacing, sag rods, girts or small simple members, a load more appropriate to the size and use of the part should be used. Design requirements for the installed elements as well as construction loads need to be considered when specifying minimum loads for connections.

2. Simple Connections

Simple connections are considered in Sections B3.6a. and J1.2. In Section B3.6a "simple" connections are defined (with further elaboration in the Commentary for Section B3.6) as a guide to idealization of the structure for the purpose of analysis. The assumptions made in the analysis determine the outcome of the analysis that serves as the basis for design (for connections that means the force and deformation demands that the connection must resist). Section J1.2 focuses on the actual proportioning of the connection elements to achieve the required resistance. In short, Section B3.6a establishes the modeling assumptions that determine the design forces and deformations for use in Section J1.2.

Sections B3.6a and J1.2 are not mutually exclusive. If a "simple" connection is assumed for analysis, the actual connection, as finally designed, must deliver performance consistent with that assumption. For a simple connection it is important to verify that the performance is consistent with the design assumptions; in other words, the connection must be able to meet the required rotation and must not introduce strength and stiffness that significantly alter the mode of response.

3. Moment Connections

Two types of moment connections are defined in Section B3.6b: fully restrained (FR) and partially restrained (PR). FR moment connections must have sufficient strength and stiffness to transfer moment and maintain the angle between connected members. PR moment connections are designed to transfer moments but also allow rotation between connected members as the loads are resisted. The response characteristics of a PR connection must be documented in the technical literature or established by analytical or experimental means. The component elements of a PR connection must have sufficient strength, stiffness and deformation capacity to satisfy the design assumptions.

4. Compression Members with Bearing Joints

The provisions for "compression members other than columns finished to bear" are intended to account for member out-of-straightness and also to provide a degree of robustness in the structure so as to resist unintended or accidental lateral loadings that may not have been considered explicitly in the design.

A provision analogous to that in Section J1.4(b)(i), requiring that splice materials and connectors have an available strength of at least 50 percent of the required compressive strength, has been in the AISC Specifications for more than 40 years. The current Specification clarifies this requirement by stating that the force for proportioning the splice materials and connectors is a tensile force. This avoids uncertainty as to how to handle situations where compression on the connection imposes no force on the connectors.

Proportioning the splice materials and connectors for 50 percent of the required member strength is simple, but can be very conservative. In Section J1.4(b)(ii), the Specification offers an alternative that addresses directly the design intent of these provisions. The lateral load of 2 percent of the required compressive strength of the member simulates the effect of a kink at the splice, caused by an end finished slightly out-of-square or other construction condition. Proportioning the connection for the resulting moment and shear also provides a degree of robustness in the structure.

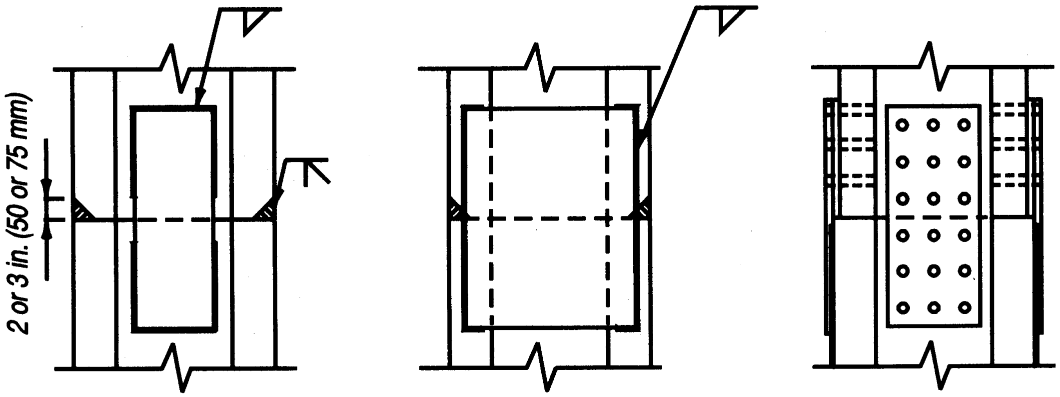
5. Splices in Heavy Sections

Solidified but still hot filler metal contracts significantly as it cools to ambient temperature. Shrinkage of large groove welds between elements that are not free to move so as to accommodate the shrinkage causes strains in the material adjacent to the weld that can exceed the yield point strain. In thick material the weld shrinkage is restrained in the thickness direction, as well as in the width and length directions, causing triaxial stresses to develop that may inhibit the ability to deform in a ductile manner. Under these conditions, the possibility of *brittle fracture* increases.

When splicing hot-rolled shapes with flange thicknesses exceeding 2 in. (50 mm) or heavy welded built-up members, these potentially harmful weld shrinkage strains can be avoided by using bolted splices, fillet-welded lap splices, or splices that combine a welded and bolted detail (see Figure C-J1.1). Details and techniques that perform well for materials of modest thickness usually must be changed or supplemented by more demanding requirements when welding thick material. The provisions of AWS D1.1 (AWS, 2004) are minimum requirements that apply to most structural welding situations; however, when designing and fabricating welded splices of hot-rolled shapes with flange thicknesses exceeding 2 in. (50 mm) and similar built-up cross sections, special consideration must be given to all aspects of the welded splice detail:

(1) Notch-toughness requirements should be specified for tension members; see Commentary Section A3.
(2) Generously sized weld access holes (see Section J1.6) are required to provide increased relief from concentrated weld shrinkage strains, to avoid close juncture of welds in orthogonal directions, and to provide adequate clearance for the exercise of high quality workmanship in hole preparation, welding and for ease of inspection.
(3) Preheating for thermal cutting is required to minimize the formation of a hard surface layer.
(4) Grinding of copes and access holes to bright metal to remove the hard surface layer is required, along with inspection using magnetic particle or dye-penetrant methods, to verify that transitions are free of notches or cracks.

In addition to tension splices of truss chord members and tension flanges of flexural members, other joints fabricated from heavy sections subject to tension should be given special consideration during design and fabrication.



(a) Shear plate welded to web

(b) Shear plate welded to flange tips

(c) Bolted splice plates

Fig. C-J1.1. Alternative splices that minimize weld restraint tensile stresses.

Previous editions of this Specification mandated that backing bars and weld tabs be removed from all splices of heavy sections. These requirements were deliberately removed from this edition, being judged unnecessary and, in some situations, potentially resulting in more harm than good. The Specification still permits the engineer of record to specify their removal when this is judged appropriate.

The previous requirement for the removal of backing bars necessitated, in some situations, that such operations be performed out-of-position; that is, the welding required to restore the backgouged area had to be applied in the overhead position. This may necessitate alternate equipment for gaining access, different welding equipment, processes and/or procedures, and other practical constraints. When box sections made of plate are spliced, access to the interior side (necessary for backing removal) is typically impossible.

Weld tabs that are left in place on splices act as "short attachments" and attract little stress. Even though it is acknowledged that weld tabs might contain regions of inferior quality weld metal, the stress concentration effect is minimized since little stress is conducted through the attachment.

6. Beam Copes and Weld Access Holes

Beam copes and weld access holes are frequently required in the fabrication of structural components. The geometry of these structural details can affect the components' performance. The size and shape of beam copes and weld access holes can have a significant effect on the ease of depositing sound weld metal, the ability to conduct nondestructive examinations, and the magnitude of the stresses at the geometric discontinuities produced by these details.

Weld access holes used to facilitate welding operations are required to have a minimum length from the toe of the weld preparation (see Figure C-J1.2) equal to 1.5 times the thickness of the material in which the hole is made. This minimum length is expected to accommodate and relieve a significant amount of the weld shrinkage strains at the web-to-flange intersection.

The height of the weld access hole must provide sufficient clearance for ease of welding and inspection and must be large enough to allow the welder to deposit sound weld metal through and beyond the web. A weld access hole height equal to 1.5 times the thickness of the material with the access hole but not less than 1 in. (25 mm) has been judged to satisfy these welding and inspection requirements. The height of the weld access hole need not exceed 2 in. (50 mm).

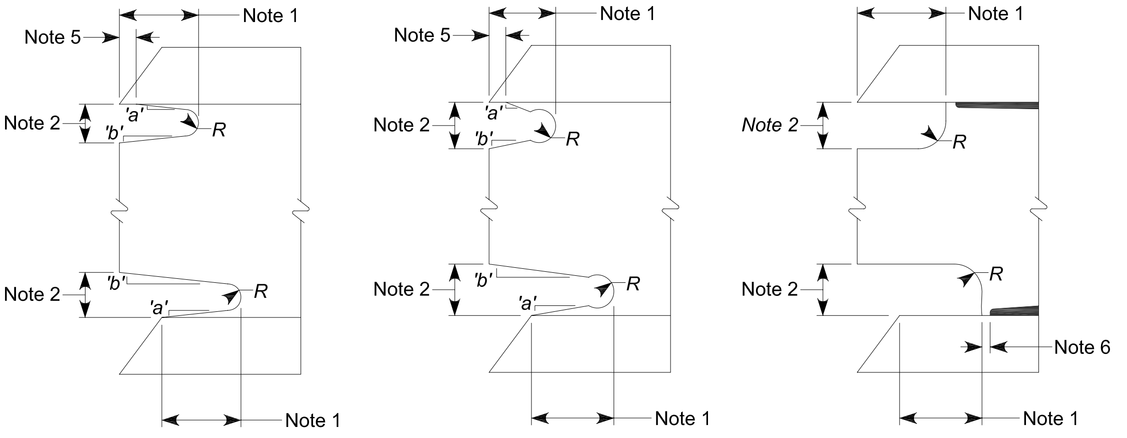
The geometry of the reentrant corner between the web and the flange determines the level of stress concentration at that location. A 90° reentrant corner having a very small radius produces a very high stress concentration that may lead to rupture of the flange. Consequently, to minimize the stress concentration at this location, the edge of the web shall be sloped or curved from the surface of the flange to the reentrant surface of the access hole.

Stress concentrations along the perimeter of beam copes and weld access holes also can affect the performance of the joint. Consequently, all beam copes and weld access holes are required to be free of stress raisers such as notches and gouges.

Stress concentrations at web-to-flange intersections of built-up shapes can be decreased by terminating the weld away from the access hole. Thus, for built-up shapes with fillet welds or partial-joint-penetration groove welds that join the web to the flange, the weld access hole may terminate perpendicular to the flange, provided that the weld is terminated at least one thickness of the web away from the access hole.

7. Placement of Welds and Bolts

Slight eccentricities between the gravity axis of single and double angle members and the center of gravity of connecting bolts or rivets have long been ignored as having negligible effect on the static strength of such members. Tests have shown that similar practice is warranted in the case of welded members in statically loaded structures (Gibson and Wake, 1942).



Alternate 1

Alternate 2

Alternate 3

Rolled Shapes and Built-Up Shapes Assembled Prior to Cutting the Access Hole

Built-Up Shapes Assembled After Cutting the Access Hole

Notes: There are typical details for joints welded from one side against steel backing. Alternative details are discussed in the commentary text.

(1) Width: Greater of $1.5 \times t_w$ or $1^1/_2$ in. (38 mm). Tolerance is $\pm\, ^1/_4$ in. (6 mm).

(2) Height: Greater of $1.5\, t_w$ or 1 in. (25 mm) but need not exceed 2 in. (50 mm).

(3) R: $^3/_8$ in. min. (8 mm). Grind the thermally cut surfaces of access holes in heavy shapes as defined in Section A3.1c and A3.1d.

(4) Slope 'a' forms a transition from the web to the flange. Slope 'b' may be horizontal.

(5) The bottom of the top flange is to be contoured to permit the tight fit of backing bars where they are to be used.

(6) The web-to-flange weld of built-up members is to be held back a distance of at least the weld size from the edge of the access hole.

Fig. C-J1.2. Weld access hole geometry.

However, the fatigue life of eccentrically loaded welded angles has been shown to be very short (Kloppel and Seeger, 1964). Notches at the roots of fillet welds are harmful when alternating tensile stresses are normal to the axis of the weld, as could occur due to bending when axial cyclic loading is applied to angles with end welds not balanced about the neutral axis. Accordingly, balanced welds are required when such members are subjected to cyclic loading (see Figure C-J1.3).

8. Bolts in Combination with Welds

As in previous editions, this Specification does not permit bolts to share the load with welds except for bolts in shear connections. The conditions for load sharing have, however, changed substantially based on recent research (Kulak and Grondin, 2001). For shear-resisting connections with longitudinally loaded fillet welds, load sharing between the longitudinal welds and bolts in standard holes or short-slotted holes transverse to the direction of the load is permitted, but the contribution of the bolts is limited to 50 percent of the available strength of the equivalent bearing-type connection. Both A307 and high-strength bolts are permitted. The heat of welding near bolts will not alter the mechanical properties of the bolts.

In making alterations to existing structures, the use of welding to resist loads other than those produced by existing dead load present at the time of making the alteration is permitted for riveted connections and high-strength bolted connections if the bolts are pretensioned to the levels in Table J3.1 or J3.1M prior to welding.

The restrictions on bolts in combination with welds do not apply to typical bolted/welded beam-to-girder and beam-to-column connections and other comparable connections (Kulak, Fisher, and Struik, 1987).

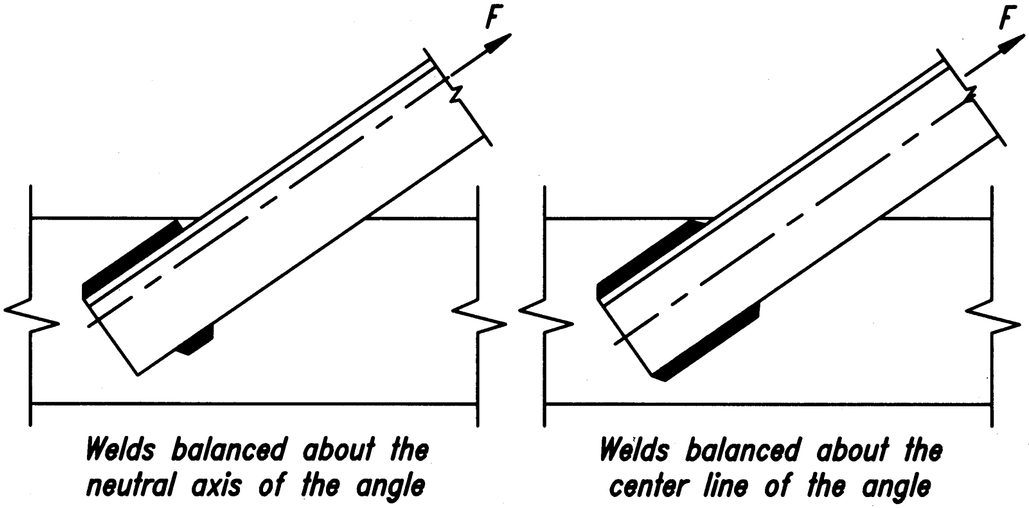


Fig. C-J1.3. Balanced welds.

9. High-Strength Bolts in Combination with Rivets

When high-strength bolts are used in combination with rivets, the ductility of the rivets permits the direct addition of the strengths of the two fastener types.

10. Limitations on Bolted and Welded Connections

Pretensioned bolts, slip-critical bolted connections, or welds are required whenever connection slip can be detrimental to the performance of the structure or there is a possibility that nuts will back off. Snug-tightened high-strength bolts are recommended for all other connections.

J2. WELDS

Selection of weld type [complete-joint-penetration (CJP) groove weld versus fillet versus partial-joint-penetration (PJP) groove weld] depends on base connection geometry (butt versus T or corner), in addition to required strength, and other issues discussed below. Consideration of notch effects and the ability to evaluate with nondestructive testing may be appropriate for cyclically loaded joints or joints expected to deform plastically.

1. Groove Welds

1a. Effective Area

Effective throats larger than those in Table J2.1 can be qualified by tests. The weld reinforcement is not used in determining the effective throat of a groove weld.

1b. Limitations

Table J2.3 provides a minimum size of PJP groove weld for a given thickness of the thinner part joined. Structural steel with a specified minimum yield stress of 50 ksi (350 MPa) is the prevalent material. The use of prequalified weld procedures is prevalent in structural welding. The minimum weld sizes required in this Specification are appropriate for filler metal prequalified with 50-ksi (350 MPa) base metal. Also, see the commentary to Section J2.2b for fillet weld limitations.

2. Fillet Welds

2a. Effective Area

The effective throat of a fillet weld is based on the root of the joint and the face of the diagrammatic weld; hence this definition gives no credit for weld penetration or reinforcement at the weld face. Some welding procedures produce a consistent penetration beyond the root of the weld. This penetration contributes to the strength of the weld. However, it is necessary to demonstrate that the weld procedure to be used produces this increased penetration. In practice, this can be done initially by cross-sectioning the runoff plates of the joint. Once this is done, no further testing is required, as long as the welding procedure is not changed.

2b. Limitations

Table J2.4 provides the minimum size of a fillet weld for a given thickness of the thinner part joined. The requirements are not based on strength considerations, but on the quench effect of thick material on small welds. Very rapid cooling of weld metal may result in a loss of ductility. Furthermore, the restraint to weld metal shrinkage provided by thick material may result in weld cracking. The use of the thinner part to determine the minimum size weld is based on the prevalence of the use of filler metal considered to be "low hydrogen." Because a $^5/_{16}$-in. (8 mm) fillet weld is the largest that can be deposited in a single pass by the SMAW process and still be considered prequalified under AWS D1.1, $^5/_{16}$ in. (8 mm) applies to all material $^3/_4$ in. (19 mm) and greater in thickness, but minimum preheat and interpass temperatures are required by AWS D1.1. Both the engineer of record and the shop welder must be governed by the requirements.

Table J2.3 gives the minimum effective throat thickness of a PJP groove weld. Notice that for PJP groove welds Table J2.3 goes up to a plate thickness of over 6 in. (150 mm) and a minimum weld throat of $^5/_8$ in. (16 mm), whereas for fillet welds Table J2.4 goes up to a plate thickness of over $^3/_4$ in. (19 mm) and a minimum leg size of fillet weld of only $^5/_{16}$ in. (8 mm). The additional thickness for PJP groove welds is intended to provide for reasonable proportionality between weld and material thickness.

For thicker members in lap joints, it is possible for the welder to melt away the upper corner, resulting in a weld that appears to be full size but actually lacks the required weld throat dimension. See Figure C-J2.1(a). On thinner members, the full weld throat is likely to be achieved, even if the edge is melted away.

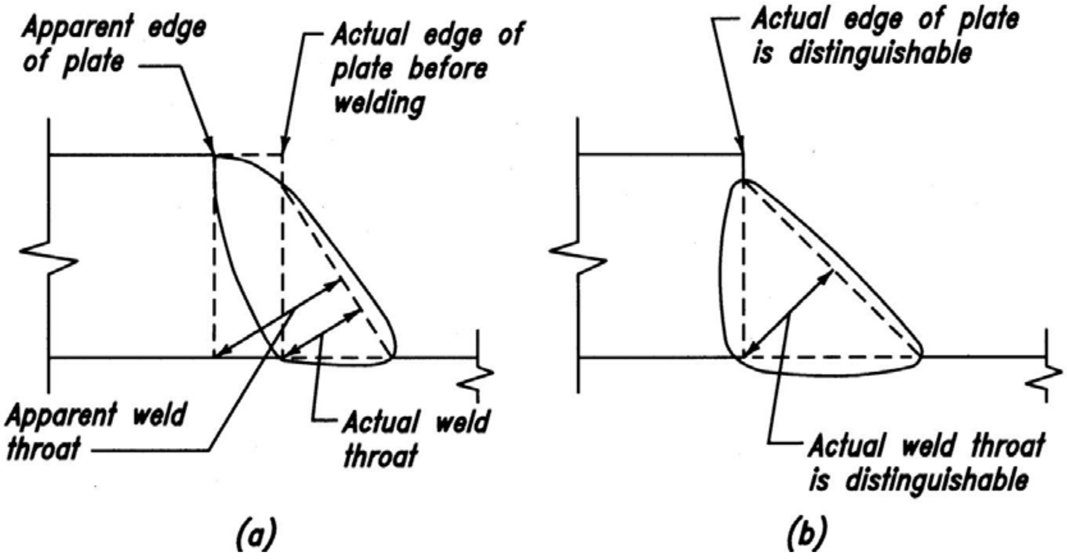


Fig. C-J2.1. Identification of plate edge.

Accordingly, when the plate is $^1/_4$ in. (6 mm) or thicker, the maximum fillet weld size is $^1/_{16}$ in. (2 mm) less than the plate thickness, ensuring that the edge remains behind [see Figure C-J2.1(b)].

Where longitudinal fillet welds are used alone in a connection (see Figure C-J2.2), Section J2.2b requires that the length of each weld be at least equal to the width of the connecting material because of shear lag (Freeman, 1930).

By providing a minimum lap of five times the thickness of the thinner part of a lap joint, the resulting rotation of the joint when pulled will not be excessive, as shown in Figure C-J2.3. Fillet welded lap joints under tension tend to open and apply a tearing action at the root of the weld as shown in Figure C-J2.4(b), unless restrained by a force F as shown in Figure C-J2.4(a).

End returns are not essential for developing the capacity of fillet welded connections and have a negligible effect on their strength. Their use has been encouraged to ensure that the weld size is maintained over the length of the weld, to enhance the fatigue resistance of cyclically loaded flexible end connections, and to increase the plastic deformation capability of such connections.

The weld capacity database on which the specifications were developed had no end returns. This includes the study reported in Higgins and Preece (1968), the

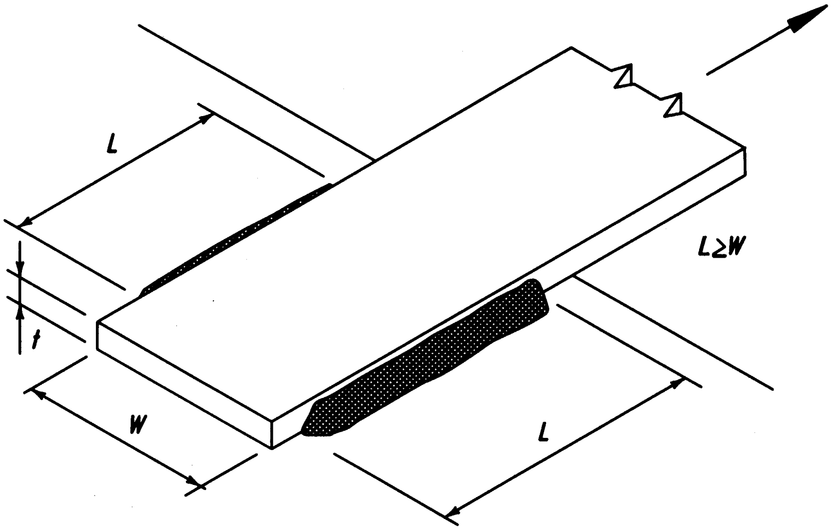


Fig. C-J2.2. Longitudinal fillet welds.

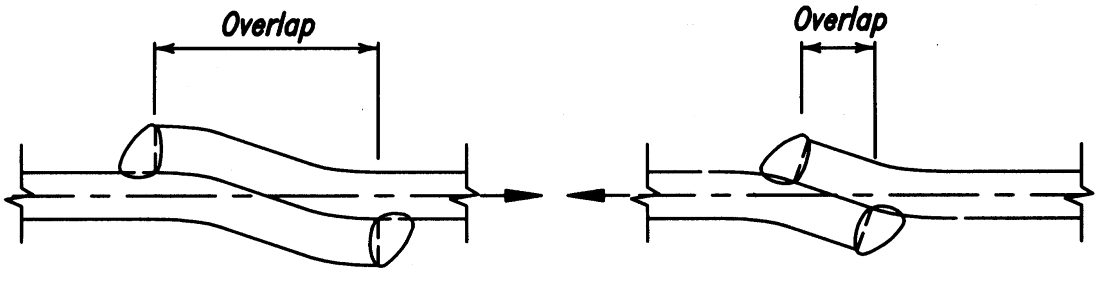


Fig. C-J2.3. Minimum lap.

seat angle tests in Lyse and Schreiner (1935), the seat and top angle tests in Lyse and Gibson (1937), the tests on beam webs welded directly to a column or girder by fillet welds in Johnston and Deits (1942), and the tests on eccentrically loaded welded connections reported in Butler, Pal, and Kulak (1972). Hence, the current strength values and joint-capacity models do not require end returns when the required weld size is provided. Johnston and Green (1940) noted that movement consistent with the design assumption of no end restraint (in other words, joint flexibility) was enhanced without end returns. They also verified that greater plastic deformation of the connection was achieved when end returns existed, although the strength was not significantly different.

When longitudinal fillet welds parallel to the stress are used to transmit the load to the end of an axially loaded member, the welds are termed "end loaded." Typical examples of such welds include, but are not limited to: (a) longitudinally welded lap joints at the end of axially loaded members; (b) welds attaching bearing stiffeners; and (c) similar cases. Typical examples of longitudinally loaded fillet welds are not considered end loaded include, but are not limited to: (a) welds that connect plates or shapes to form built-up cross sections in which the shear force is applied to each increment of length of weld depending upon the distribution of the shear along the length of the member; and (b) welds attaching beam web connection angles and shear plates because the flow of shear force from the beam or girder web to the weld is essentially uniform throughout the weld length; that is, the weld is not end-loaded despite the fact that it is loaded parallel to the weld axis. Neither does the reduction coefficient, β, apply to welds attaching stiffeners to webs because the stiffeners and welds are not subject to calculated axial stress but merely serve to keep the web flat.

The distribution of stress along the length of end-loaded fillet welds is not uniform and is dependent upon complex relationships between the stiffness of the longitudinal fillet weld relative to the stiffness of the connected materials. Experience has shown that when the length of the weld is equal to approximately 100 times the weld size or less, it is reasonable to assume the effective length is equal to or less than the actual length. For weld lengths greater than 100 times the weld size, the effective length should be taken less than the actual length. The reduction coefficient, β, provided in Section J2.2b is the equivalent to that given in Eurocode 3 (1992), which is a simplified approximation of exponential

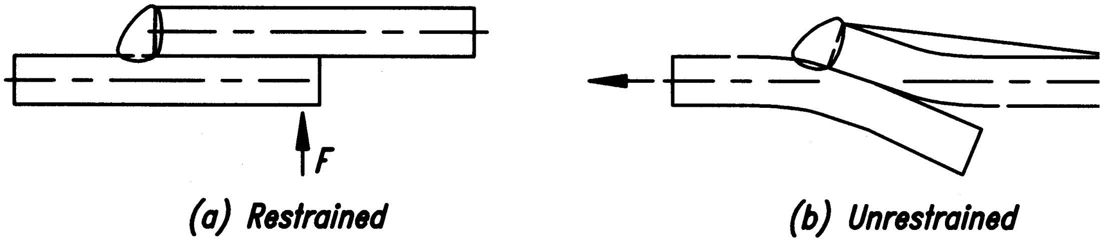


Fig. C-J2.4. Restraint of lap joints.

formulas developed by finite element studies and tests preformed in Europe over many years. The provision is based on the combined consideration of the nominal strength for fillet welds with leg size less than $^1/_4$ in. (6 mm) and of a judgment-based serviceability limit of slightly less than $^1/_{32}$ in. (1 mm) displacement at the end of the weld for welds with leg size $^1/_4$ in. (6 mm) and larger. The mathematical form of the β factor implies that the minimum strength of an end-loaded weld is achieved when the length is approximately 300 times the leg size. Because it is illogical to conclude that the total strength of a weld longer than 300 times the weld size is more than that of a shorter weld, the length reduction coefficient is taken as 0.6 when the weld length is greater than 300 times the leg size.

In most cases, fillet weld terminations do not affect the strength or serviceability of connections. However, in certain cases the disposition of welds affect the planned function of the connection, and notches may affect the static strength and/or the resistance to crack initiation if *cyclic loads* of sufficient magnitude and frequency occur. For these cases, terminations before the end of the joint are specified to provide the desired profile and performance. In cases where profile and notches are less critical, terminations are permitted to be run to the end. In most cases, stopping the weld short of the end of the joint will not reduce the strength of the weld. The small loss of weld area due to stopping the weld short of the end of the joint by one to two weld sizes is not typically considered in the calculation of weld strength. Only short weld lengths will be significantly affected by this.

The following situations require special attention:

(1) For lapped joints where one part extends beyond the end or edge of the part to which it is welded and if the parts are subject to calculated tensile stress at the start of the overlap, it is important that the weld terminate a short distance from the stressed edge. For one typical example, the lap joint between the tee chord and the web members of a truss, the weld should not extend to the edge of the tee stem (see Figure C-J2.5). The best technique to avoid inadvertent notches at this critical location is to strike the welding arc at a point slightly back from the edge and proceed with welding in the direction away from the edge (see Figure C-J2.6). Where framing angles extend beyond the end of the beam web to which they are welded, the free end of the beam web is subject to zero stress; thus, it is permissible for the fillet weld to extend continuously across the top end, along the side and along the bottom end of the angle to the extreme end of the beam (see Figure C-J2.7).

(2) For connections such as framing angles and framing tees, which are assumed in the design of the structure to be *flexible connections*, the top and bottom edges of the outstanding legs or flanges must be left unwelded over a substantial portion of their length to assure flexibility of the connection. Tests have shown that the static strength of the connection is the same with or without end returns; therefore the use of returns is optional, but if used, their length must be restricted to not more than four times the weld size (Johnston and Green, 1940) (see Figure C-J2.8).

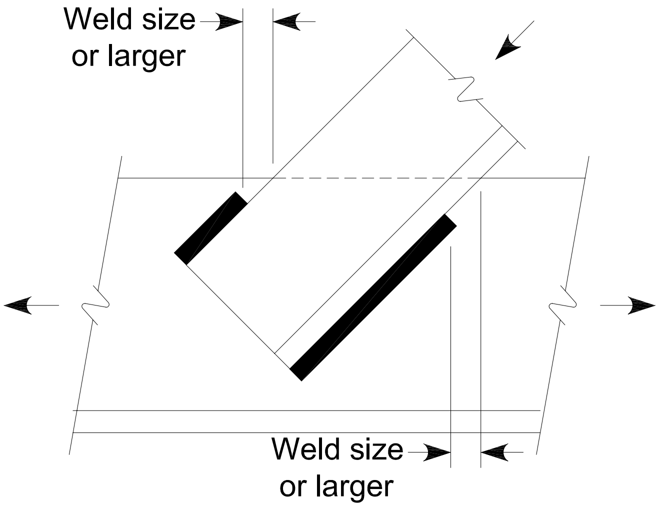


Fig. C-J2.5. Fillet welds near tension edges.

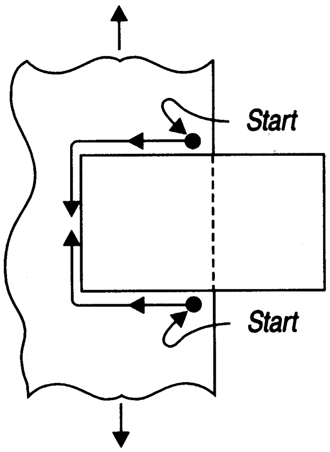


Fig. C-J2.6. Suggested direction of welding travel to avoid notches.

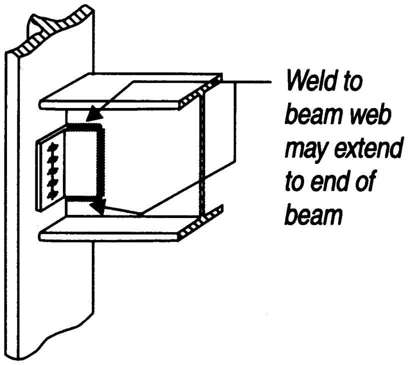


Fig. C-J2.7. Fillet weld details on framing angles.

(3) Experience has shown that when ends of intermediate transverse stiffeners on the webs of plate girders are not welded to the flanges (the usual practice), small torsional distortions of the flange occur near shipping bearing points in the normal course of shipping by rail or truck and may cause high out-of-plane bending stresses (up to the yield point) and fatigue cracking at the toe of the web-to-flange welds. This has been observed even with closely fitted stiffeners. The intensity of these out-of-plane stresses may be effectively limited and cracking prevented if "breathing room" is provided by terminating the stiffener weld away from the web-to-flange welds. The unwelded distance should not exceed six times the web thickness so that column buckling of the web within the unwelded length does not occur.

(4) For fillet welds that occur on opposite sides of a common plane, it is difficult to deposit a weld continuously around the corner from one side to the other without causing a gouge in the corner of the parts joined; therefore the welds must be interrupted at the corner (see Figure C-J2.9).

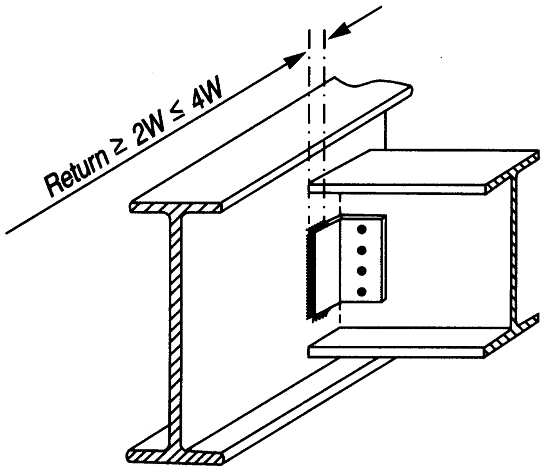


Fig. C-J2.8. Flexible connection returns optimal unless subject to fatigue.

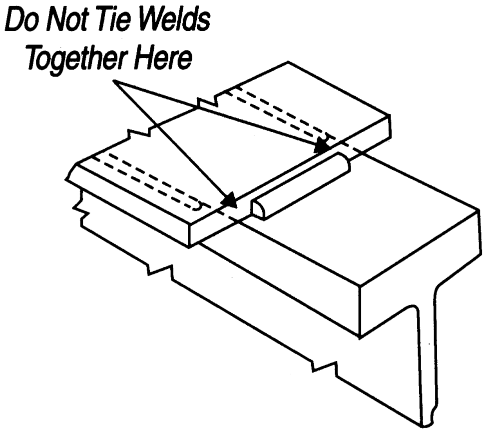


Fig. C-J2.9. Details for fillet welds that occur on opposite sides of a common plane.

3. Plug and Slot Welds

A plug weld is a weld made in a circular hole in one member of a joint fusing that member to another member. Both plug and slot welds are only applied to lap joints. Care should be taken when plug or slot welds are applied to structures subject to cyclic loading as the fatigue performance of these welds is limited. A slot weld is a weld made in an elongated hole in one member of a joint fusing that member to another member. A fillet weld inside a hole or slot is not a plug weld. A “puddle weld”, typically used for joining decking to the supporting steel, is not the same as a plug weld.

3a. Effective Area

When plug and slot welds are detailed in accordance with Section J2.3b, the strength of the weld is controlled by the size of the fused area between the weld and the base metal. The total area of the hole or slot is used to determine the effective area.

3b. Limitations

Plug and slot welds are limited to situations where they are loaded in shear, or where they are used to prevent elements of a cross section from buckling, such as for web doubler plates on deeper rolled sections. Plug and slot welds are only allowed where the applied loads result in shear between the joined materials—they are not to be used to resist direct tensile loads.

The geometric limitations on hole and slot sizes are prescribed in order to provide a geometry that is conducive to good fusion. Deep, narrow slots and holes make it difficult for the welder to gain access and see the bottom of the cavity into which weld metal must be placed. Where access is difficult, fusion may be limited, and the strength of the connection reduced.

4. Strength

The strength of welds is governed by the strength of either the base material or the deposited weld metal. Table J2.5 presents the nominal weld strengths and the ϕ and Ω factors, as well as the limitations on filler metal strength levels.

The strength of a joint that contains a complete-joint-penetration (CJP) groove weld, whether loaded in tension or compression, is dependent upon the strength of the base metal, and no computation of the strength of the CJP groove weld is required. For tension applications, matching strength filler metal is required, as defined in AWS D1.1 Table 3.1. For compression applications, up to a 10 ksi (70 MPa) decrease in filler metal strength is permitted, which is equivalent to one strength level.

CJP groove welds loaded in tension or compression parallel to the weld axis, such as for the groove welded corners of box columns, do not transfer primary loads across the joint. In cases such as this, no computation of the strength of the CJP groove weld strength is required.

CJP groove welded tension joints are intended to provide strength equivalent to the base metal, therefore matching filler metal is required. CJP groove welds have been shown not to exhibit compression failure even when they are undermatched. The amount of undermatching before unacceptable deformation occurs has not been established, but one standard strength level is conservative and therefore permitted. Joints in which the weld strength is calculated based on filler metal classification strength can be designed using any filler metal strength equal to or less than matching. Filler metal selection is still subject to compliance with AWS D1.1.

The nominal strength of partial-joint-penetration (PJP) groove welded joints in compression is higher than for other joints because compression limit states are not observed on weld metal until significantly above the yield strength.

Connections that contain PJP groove welds designed to bear in accordance with Section J1.4(b), and where the connection is loaded in compression, are not limited in capacity by the weld since the surrounding base metal can transfer compression loads. When not designed in accordance with Section J1.4(b), an otherwise similar connection must be designed considering the possibility that either the weld or the base metal may be the critical component in the connection.

The factor of 0.6 on F_{EXX} for the tensile strength of PJP groove welds is an arbitrary reduction that has been in effect since the early 1960s to compensate for the notch effect of the unfused area of the joint, uncertain quality in the root of the weld due to the inability to perform nondestructive evaluation, and the lack of a specific notch-toughness requirement for filler metal. It does not imply that the tensile failure mode is by shear stress on the effective throat, as in fillet welds.

Column splices have historically been connected with relatively small PJP groove welds. Frequently, erection aids are available to resist construction loads. Columns are intended to be in contact bearing in splices and on base plates. Section M4.4 recognizes that, in the as-fitted product, the contact may not be consistent across the joint and therefore provides rules assuring some contact that limits the potential deformation of weld metal and the material surrounding it. These welds are intended to hold the columns in place, not to transfer the compressive loads. Additionally, the effects of very small deformation in column splices are accommodated by normal construction practices. Similarly, the requirements for base plates and normal construction practice assure some bearing at bases. Therefore the compressive stress in the weld metal does not need to be considered as the weld metal will deform and subsequently stop when the columns bear. Other PJP groove welded joints connect members that may be subject to unanticipated loads and may fit with a gap. Where these connections are finished to bear, fit-up may not be as good as that specified in Section M4.4 but some bearing is anticipated so the weld is to be designed to resist loads defined in Section J1.4(b) using the factors, strengths and effective areas in Table J2.5. Where the joints connect members that are not finished to bear, the welds are designed for the total required load using the available strengths, and areas in Table J2.5.

In Table J2.5 the nominal strength of fillet welds is determined from the effective throat area, whereas the strength of the connected parts is governed by their respective thicknesses. Figure C-J2.10 illustrates the shear planes for fillet welds and base material:

(1) Plane 1-1, in which the strength is governed by the shear strength of the material A.
(2) Plane 2-2, in which the strength is governed by the shear strength of the weld metal.
(3) Plane 3-3, in which the strength is governed by the shear strength of the material B.

The strength of the welded joint is the lowest of the strengths calculated in each plane of shear transfer. Note that planes 1-1 and 3-3 are positioned away from the fusion areas between the weld and the base material. Tests have demonstrated that the stress on this fusion area is not critical in determining the shear strength of fillet welds (Preece, 1968).

The shear planes for plug and PJP groove welds are shown in Figure C-J2.11 for the weld and base metal. Generally the base metal will govern the shear strength.

When weld groups are loaded in shear by an external load that does not act through the center of gravity of the group, the load is eccentric and will tend to cause a relative rotation and translation between the parts connected by the weld. The point about which rotation tends to take place is called the instantaneous center of rotation. Its location is dependent upon the load eccentricity, geometry of the weld group, and deformation of the weld at different angles of the resultant elemental force relative to the weld axis.

The individual strength of each unit weld element can be assumed to act on a line perpendicular to a ray passing through the instantaneous center and that element's location (see Figure C-J2.12).

The ultimate shear strength of weld groups can be obtained from the load deformation relationship of a single-unit weld element. This relationship was originally

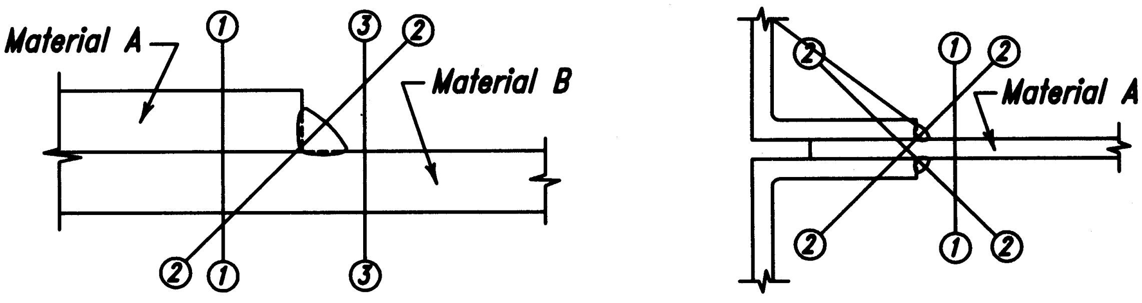
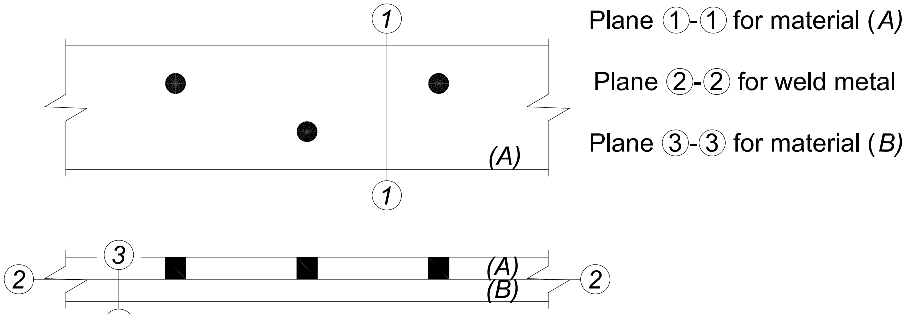


Fig. C-J2.10. Shear planes for fillet welds loaded in longitudinal shear.

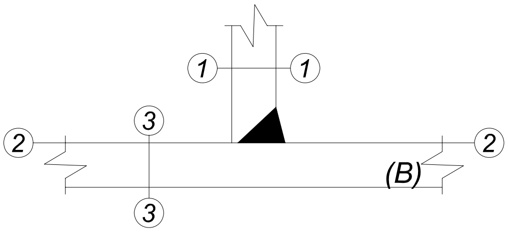
given in Butler and others (1972) for E60 (E43) electrodes. Curves for E70 (E48) electrodes were reported in Lesik and Kennedy (1990).

Unlike the load-deformation relationship for bolts, strength and deformation performance in welds are dependent on the angle that the resultant elemental force makes with the axis of the weld element as shown in Figure C-J2.12. The actual load deformation relationship for welds is given in Figure C-J2.13, taken from Lesik and Kennedy (1990). Conversion of the SI equation to U.S. customary units results in the following weld strength equation for R_n:

$$R_n = 0.852(1.0 + 0.50 \sin^{1.5}\theta)F_{EXX}A_w \qquad \text{(C-J2-1)}$$



(a) Plug welds



(b) Partial-joint-penetration groove welds

Fig. C-J2.11. Shear planes for plug and PJP groove welds.

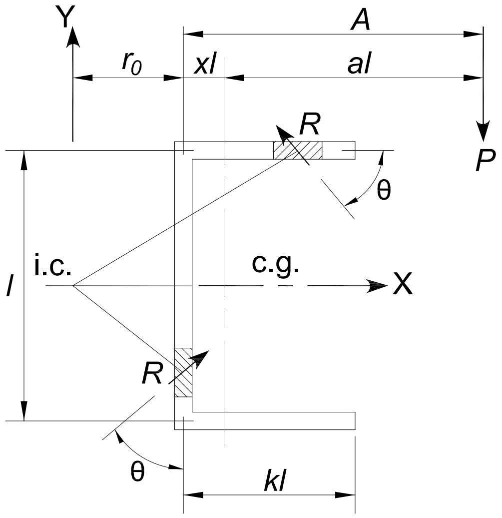


Fig. C-J2.12. Weld element nomenclature.

Because the maximum strength is limited to $0.60F_{EXX}$ for longitudinally loaded welds ($\theta = 0°$), the Specification provision provides, in the reduced equation coefficient, a reasonable margin for any variation in welding techniques and procedures. To eliminate possible computational difficulties, the maximum deformation in the weld elements is limited to $0.17w$. For design convenience, a simple elliptical formula is used for $f(p)$ to closely approximate the empirically derived polynomial in Lesik and Kennedy (1990).

The total strength of all the weld elements combine to resist the eccentric load and, when the correct location of the instantaneous center has been selected, the three in-plane equations of statics ($\Sigma F_x = 0$, $\Sigma F_y = 0$, $\Sigma M = 0$) will be satisfied. Numerical techniques, such as those given in Brandt (1982), have been developed to locate the instantaneous center of rotation subject to convergent tolerances.

5. Combination of Welds

When determining the capacity of a combination PJP groove weld and fillet weld contained within the same joint, the total throat dimension is not the simple addition of the fillet weld throat and the groove weld throat. In such cases, the resultant throat of the combined weld (dimension from root perpendicular to face of fillet weld) must be determined and the design based upon this dimension.

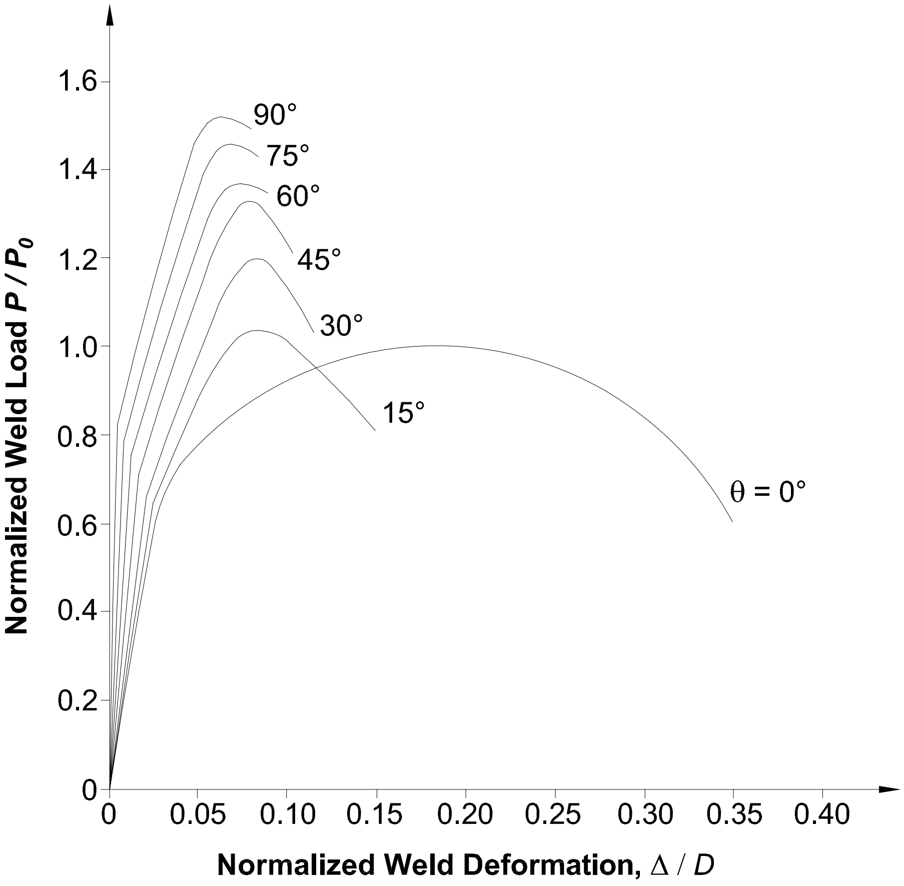


Fig. C-J2.13. Load deformation relationship.

6. Filler Metal Requirements

Applied and *residual stresses* and geometrical discontinuities from backup bars with associated notch effects contribute to sensitivity to fracture. Additionally, some weld metals in combination with certain procedures result in welds with low notch toughness. Accordingly, this Specification requires a minimum specified toughness for weld metals in those joints that are subject to more significant applied stresses and toughness demands. The level of toughness required is selected as one level more conservative than the base metal requirement for hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm).

7. Mixed Weld Metal

Problems can occur when incompatible weld metals are used in combination and notch-tough composite weld metal is required. For instance, tack welds deposited using a self-shielded process with aluminum deoxidizers in the electrodes and subsequently covered by SAW weld passes can result in a composite weld metal with low notch-toughness, despite the fact that each process by itself could provide notch-tough weld metal.

Potential concern about intermixing weld metal types is limited to situations where one of the two weld metals is deposited by the self-shielded flux-cored arc welding (FCAW-s) process. Changes in tensile and elongation properties have been demonstrated to be of insignificant consequence. Notch toughness is the property that can be affected the most. Many compatible combinations of FCAW-s and other processes are commercially available.

J3. BOLTS AND THREADED PARTS

1. High-Strength Bolts

In general, the use of high-strength bolts is required to conform to the provisions of the *Specification for Structural Joints Using ASTM A325 or A490 Bolts* (RCSC, 2004) as approved by the Research Council on Structural Connections. Kulak (2002) provides an overview of the properties and use of high-strength bolts.

Occasionally the need arises for the use of high-strength bolts of diameters and lengths in excess of those available for ASTM A325 or A325M and ASTM A490 or A490M bolts. For joints requiring diameters in excess of $1^1/_2$ in. (38 mm) or lengths in excess of about 8 in. (200 mm), Section J3.1 permits the use of ASTM A449 bolts and ASTM A354 Grade BC and BD threaded rods. Note that anchor rods are more preferably specified as ASTM F1554 material.

Snug-tight installation is permitted for static applications involving only ASTM A325 or A325M bolts in tension or combined shear and tension. Two studies have been conducted to investigate possible reductions in strength because of varying levels of pretension in bolts within the same connection. The first investigation focused on nine, two-bolt tee stubs connected in a back-to-back configuration using $^3/_4$-in. diameter A325 bolts (Johnson, 1996). The bolt pretensions were varied from pretensioned to snug tight to finger tight. No significant loss of strength was noted as compared to the case with both fasteners pretensioned—even with one fastener pretensioned and the other finger tight. The second study tested 32 additional two-bolt tee stubs but considered both ASTM A325 and A490 fasteners and two, four-bolt tee stubs (Amrine and Swanson, 2004). The study found that no significant loss of strength resulted from having different pretensions in bolts within the same connection, even with ASTM A490 fasteners.

There are practical cases in the design of structures where slip of the connection is desirable to allow for expansion and contraction of a joint in a controlled manner. Regardless of whether force transfer is required in the direction normal to the slip direction, the nuts should be hand-tightened with a spud wrench and then backed off one-quarter turn. Furthermore, it is advisable to deform the bolt threads or use a locking nut or jamb nut to ensure that the nut does not back off further under service conditions. Thread deformation is commonly accomplished with a cold chisel and hammer applied at one location. Note that tack-welding of the nut to the bolt threads is not recommended.

2. Size and Use of Holes

To provide some latitude for adjustment in plumbing a frame during erection, three types of enlarged holes are permitted, subject to the approval of the designer. The nominal maximum sizes of these holes are given in Table J3.3 or J3.3M. The use of these enlarged holes is restricted to connections assembled with bolts and is subject to the provisions of Sections J3.3 and J3.4.

3. Minimum Spacing

The minimum spacing dimensions of $2^2/_3$ times and 3 times the nominal diameter are to facilitate construction and do not necessarily satisfy the bearing and tearout strength requirements in Section J3.10.

4. Minimum Edge Distance

The minimum edge distances given in Table J3.4 and Table J3.4M are to facilitate construction and do not necessarily satisfy the bearing and tearout strength requirements in Section J3.10. Lesser values are permitted if the requirements of Section J3.10 are satisfied.

5. Maximum Spacing and Edge Distance

Limiting the edge distance to not more than 12 times the thickness of an outside connected part, but not more than 6 in. (150 mm), is intended to provide for the exclusion of moisture in the event of paint failure, thus preventing corrosion between the parts that might accumulate and force these parts to separate. More restrictive limitations are required for connected parts of unpainted weathering steel exposed to atmospheric corrosion.

6. Tension and Shear Strength of Bolts and Threaded Parts

Tension loading of fasteners is usually accompanied by some bending due to the deformation of the connected parts. Hence, the resistance factor, ϕ, and the safety factor, Ω, are relatively conservative. The nominal tensile stress values in Table J3.2 were obtained from the equation

$$F_{nt} = 0.75F_u \qquad \text{(C-J3-2)}$$

The factor of 0.75 included in this equation accounts for the approximate ratio of the effective area of the threaded portion of the bolt to the area of the shank of the bolt for common sizes. Thus A_b is defined as the area of the unthreaded body of the bolt and the value reported for F_{nt} in Table J3.2 is calculated as 0.75 F_u.

The tensile strength given by Equation C-J3-2 is independent of whether the bolt was initially installed pretensioned or snug-tightened. Recent tests confirm that the performance of ASTM A325 and A325M bolts in tension not subjected to fatigue are unaffected by the original installation condition (Amrine and Swanson, 2004; Johnson, 1996; Murray, Kline, and Rojani, 1992). While the equation was developed for bolted connections, it was also conservatively applied to threaded parts (Kulak and others, 1987).

For ASTM A325 or A325M bolts, no distinction is made between small and large diameters, even though the minimum tensile strength, F_u, is lower for bolts with diameters in excess of 1 in. (24 mm). It was felt that such a refinement was not justified, particularly in view of the conservative resistance factor, ϕ, and safety factor, Ω, the increasing ratio of tensile area to gross area, and other compensating factors.

The values of nominal shear stress in Table J3.2 were obtained from the following equations:

$$F_{nv} = 0.50F_u, \text{ when threads are excluded from the shear planes} \qquad \text{(C-J3-3)}$$

$$F_{nv} = 0.40F_u, \text{ when threads are not excluded from the shear plane} \qquad \text{(C-J3-4)}$$

The factors 0.50 and 0.40 account for the effect of shear and for the reduced area of the threaded portion of the fastener when the threads are not excluded from the shear plane. When determining the shear strength of a fastener, the area, A_b, is multiplied by the number of shear planes. While developed for bolted connections, the equations were also conservatively applied to threaded parts. The value given for ASTM A307 bolts was obtained from Equation C-J3-4 but is specified for all cases regardless of the position of threads.

In connections consisting of only a few fasteners, the effect of differential strain on the shear in bearing fasteners is negligible (Kulak and others, 1987; Fisher, Galambos, Kulak, and Ravindra, 1978). In longer joints, the differential strain produces an uneven distribution of load between fasteners, those near the end taking a disproportionate part of the total load, so that the maximum strength per fastener is reduced. This Specification does not limit the length but requires a 20 percent reduction in shear strength for connections longer than 50 in. (1.2 m). The resistance factor, ϕ, and the safety factor, Ω, for shear in bearing-type connections already accommodate the effects of differential strain in connections less than 50 in. (1.2 m) in length. The above discussion is primarily applicable to end-loaded connections, but is applied to all connections to maintain simplicity.

Additional information regarding the development of the provisions in this section can be found in the Commentary to the RCSC *Specification* (RCSC, 2004).

7. Combined Tension and Shear in Bearing-Type Connections

Tests have shown that the strength of bearing fasteners subject to combined shear and tension resulting from externally applied forces can be closely defined by an ellipse (Kulak and others, 1987). The relationship is expressed as

$$\left(\frac{f_t}{\phi F_{nt}}\right)^2 + \left(\frac{f_v}{\phi F_{nv}}\right)^2 = 1 \qquad \text{(LRFD)} \qquad \text{(C-J3-5a)}$$

$$\left(\frac{\Omega f_t}{F_{nt}}\right)^2 + \left(\frac{\Omega f_v}{F_{nv}}\right)^2 = 1 \qquad \text{(ASD)} \qquad \text{(C-J3-5b)}$$

In these equations, f_v and f_t are the required shear stress and tensile stress, respectively, and F_{nv} and F_{nt} are the nominal shear and tensile stresses, respectively. The elliptical relationship can be replaced, with only minor deviations, by three straight lines as shown in Figure C-J3.1. The sloped portion of the straight-line representation is

$$\left(\frac{f_t}{\phi F_{nt}}\right) + \left(\frac{f_v}{\phi F_{nv}}\right) = 1.3 \qquad \text{(LRFD)} \qquad \text{(C-J3-6a)}$$

$$\left(\frac{\Omega f_t}{F_{nt}}\right) + \left(\frac{\Omega f_v}{F_{nv}}\right) = 1.3 \qquad \text{(ASD)} \qquad \text{(C-J3-6b)}$$

which results in Equations J3-3a and J3-3b.

This latter representation offers the advantage that no modification of either type of stress is required in the presence of fairly large magnitudes of the other type. Note that Equations J3-3a and J3-3b can be rewritten so as to find the nominal shear strength per unit area, F'_{nv}, as a function of the required tensile stress, f_t. These formulations are

$$F'_{nt} = 1.3F_{nt} - \frac{F_{nt}}{\phi F_{nv}} f_v \le F_{nv} \qquad \text{(LRFD)} \qquad \text{(C-J3-7a)}$$

$$F'_{nt} = 1.3F_{nt} - \frac{\Omega F_{nt}}{F_{nv}} f_v \le F_{nt} \qquad \text{(ASD)} \qquad \text{(C-J3-7b)}$$

The linear relationship was adopted for use in Section J3.7; generally, use of the elliptical relationship is acceptable (see Figure C-J3.1). A similar formulation using the elliptical solution is

$$F'_{nt} = F_{nt}\sqrt{1-\left(\frac{f_v}{\phi F_{nv}}\right)^2} \le F_{nt} \qquad \text{(LRFD)} \qquad \text{(C-J3-8a)}$$

$$F'_{nt} = F_{nt}\sqrt{1-\left(\frac{\Omega f_{nv}}{F_{nv}}\right)^2} \le F_{nt} \qquad \text{(ASD)} \qquad \text{(C-J3-8b)}$$

8. High-Strength Bolts in Slip-Critical Connections

Connections should be classified as slip-critical only when the slip is deemed by the engineer of record to affect the serviceability of the structure by excessive distortion or cause a reduction in strength or stability even though the available strength of the connection is adequate. For example, connections subject to fatigue and connections with oversized holes or slots parallel to the direction of load should be designed as slip-critical. Most connections with standard holes can be designed as bearing-type connections without concern for serviceability. For connections with three or more bolts in standard holes or slots perpendicular to the direction of force, the freedom to slip generally does not exist because one or more of the bolts are in bearing before the load is applied.

Slip resistance in bolted connections has traditionally been viewed as a serviceability limit state and these connections have been designed to resist slip due to load effects from serviceability combinations and checked as bearing connections due to load effects from strength load combinations. There are conditions,

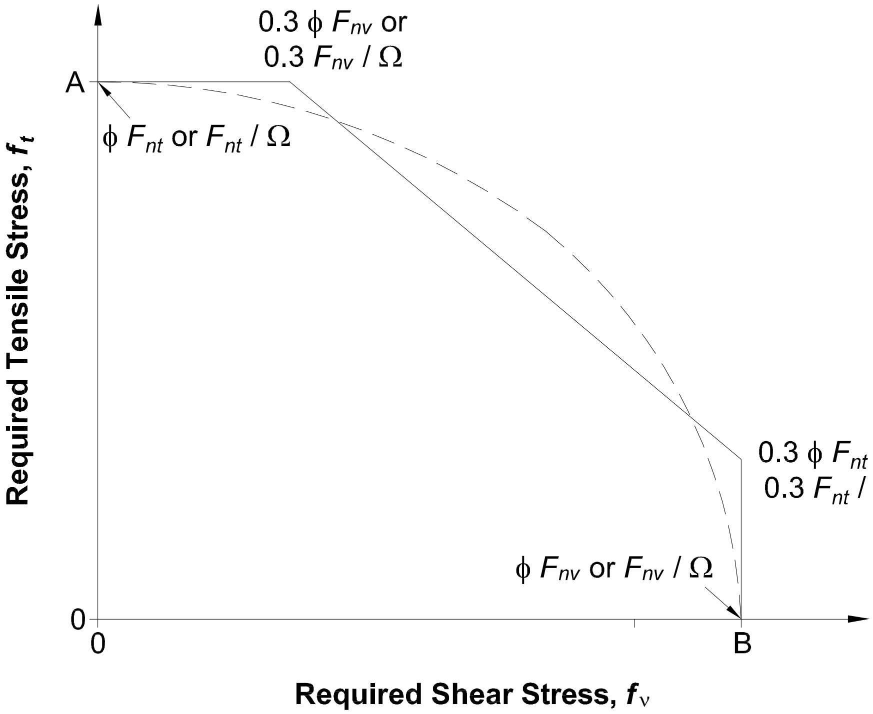


Fig. C-J3.1. Straight-line representation of elliptical solution.

however, where the deformations due to slip in connections with oversized holes and slotted holes parallel to the load could result in an increased load larger than the strength limit state. Examples where the usual assumption of serviceability-governed slip resistance may not apply are:

- High aspect ratio braced frames where the slip permitted by slots or oversized holes is relatively large and could potentially result in large P-Δ effects;
- Long-span, flat roof trusses with oversized holes, where slip could result in excessively large loads due to ponding;
- Built-up compression members where slip between the individual element ends could increase the member effective length and thus significantly reduce buckling strength;
- Any condition where the normal analysis assumption of an undeformed structure (small deflections) could be violated by connection slip resulting in increased load.

The Commentary to the 1999 *LRFD Specification* (AISC, 2000b) cautioned engineers about such conditions when utilizing long-slotted holes parallel to the direction of the load:

> If the connection is designed so that it will not slip under the effects of service loads, then the effect of the factored loads on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis. Alternatively, the connection can be designed so that it will not slip at loads up to the factored load level.

However, neither the 1999 *LRFD Specification* (AISC, 2000b) nor its Commentary provided the engineer with any guidance for designing to prevent slip at the factored load level. Since the AISC *LRFD Manual of Steel Construction*, 3rd edition (AISC, 2001) also provided two separate design aids, Tables 7-15 and 7-16, one that indicated the use of service load combinations and one that indicated the use of strength load combinations, it was sometimes believed that the use of Table 7-15 would guard against slip due to load effects from service load combinations and the use of Table 7-16 would guard against slip due to load effects from strength load combinations. These are incorrect interpretations as both tables lead to the same final result, that is, to prevent slip due to load effects from service load combinations.

The Commentary to the 1999 *LRFD Specification* (AISC, 2000b) states, "Slip of slip-critical connections is likely to occur at approximately 1.4 to 1.5 times the service load." This is based on the use of a resistance factor $\phi = 1.00$, standard holes, and calibrated wrench installation. The use of $\phi = 0.85$ for oversized and short-slotted holes and $\phi = 0.70$ for long-slotted holes perpendicular and $\phi = 0.60$ for long-slotted holes parallel to the load, increases this resistance to approximately 1.7 times the service load for oversized and short-slotted holes and even greater slip resistance for long-slotted holes. The use of the turn-of-the-nut installation method also increases slip resistance by approximately 10 to 15 percent. Hence connections with oversized and slotted holes, even when designed for the serviceability limit state provisions of the 1999 *LRFD Specification* (AISC, 2000b), will resist slip due to load effects from the strength load combinations.

Determining Required Resistance to Slip. This Specification permits all slip-critical connections with bolts in standard holes or in slotted holes perpendicular to the direction of the force to be designed as being governed by serviceability. The slight variations in geometry, which can occur due to connection slip when using this type of hole, will not change the normal analysis assumptions or result in an increase in load.

The fundamental design requirement for all connections with bolts in oversized holes and slotted holes parallel to the load is to prevent slip at the strength limit state, which conservatively assumes that the corresponding potential for change in geometry will not be negligible and that connection slip will result in significant load increase.

The engineer of record is permitted to make the determination that the effect of slip will not result in increased loads and, therefore, to design any slip-critical connection for the serviceability limit state. In either case, the design slip resistance is calculated using the load effects from either the LRFD load combinations or the ASD load combinations and the appropriate resistance factor, ϕ, or safety factor, Ω. All slip-critical connections, whether designed for the serviceability or strength limit state, must be checked for shear and bearing using the appropriate design loads.

The reliability required when designing to resist slip due to load effects from strength load combinations is subject to some interpretation. Traditionally, connection limit states require a β for bolts and fillet welds of 4.0. This is because many limit states associated with connection failure are associated with a sudden, nonductile joint separation. Since connection slip will not result in a sudden separation of the joint as long as the connection is checked as a bearing-type connection due to load effects from strength load combinations, knowing the exact level of reliability for slip resistance due to strength load combinations is not critical to connection performance. Resistance and safety factors along with the hole factors proposed for oversized holes and slotted holes approach those necessary to achieve a reliability index of 4.0. However, because of the complex factors involved in calculating the reliability of slip-critical connections and the lack of extensive statistical data on slip resistance of oversized and slotted holes, the checks for bearing and shear due to strength load combinations are required for both design methods.

Factors that Affect Slip Resistance of Joints. The following paragraphs outline the key factors affecting slip resistance in bolted steel connections:

Slip Coefficient of the Faying Surface. This Specification has combined the previous Class A and Class C surfaces into a single Class A surface category that includes unpainted clean mill scale surfaces or surfaces with Class A coatings on a blasted-cleaned surface, and hot-dip galvanized and roughened surfaces with a coefficient of friction $\mu = 0.35$. This is a slight increase in value from the previous Class A coefficient. Class B surfaces, unpainted blast-cleaned surfaces, or surfaces with Class B coatings on blast-cleaned steel remain the same at $\mu = 0.50$.

Pretensioning Method and D_u. Four bolt pretensioning methods are recognized by AISC: turn-of-the-nut, calibrated wrench, twist-off type tension-control bolt assemblies, and direct tension indicating assemblies. The mean pretension force in the bolts varies according to the method of installation. The lowest mean value is when the calibrated wrench method is used: 1.13 times the minimum specified. The turn-of-the-nut method results in a mean pretension of 1.22 to 1.35 times the minimum specified, depending on the amount the bolt is turned and the bolt grade. While the statistical information on the mean pretension level of bolts installed in the field using direct tension indicators and twist-off type tension-control bolt assemblies is limited, tests indicate they will fall somewhere between the calibrated wrench and the turn-of-the-nut method. Thus, this specification uses the minimum of these values, 1.13, for all methods of installation. This results in varying reliabilities for differing conditions. Regardless of the method used to pretension the bolts, it is important that the installation of slip-critical connections meet all of the requirements listed in the RCSC *Specification* (RCSC, 2004).

Hole Size. High-strength bolts properly installed in oversized and short-slotted holes using washers as specified in the RCSC *Specification* (RCSC, 2004) have the same resistance to slip as similar bolts in standard holes. The hole factor, $h_{sc} = 0.85$, is used to increase resistance to slip for this type of connection because of the possible consequences of increased movement with these connections. The hole factor for long-slotted holes, $h_{sc} = 0.70$, serves both to increase slip resistance for this type of connection similar to the oversized holes and to compensate for a slight loss in pretension and slip resistance due to the length of a long slot. Previous editions of the Specification had a further reduction in the hole factor, $h_{sc} = 0.60$, for slots parallel to the direction of the load. This was, in effect, a design for a strength limit state for this type of hole and the same result is achieved using the ϕ or Ω factors given in this Specification.

9. Combined Tension and Shear in Slip-Critical Connections

The slip resistance of a slip-critical connection is reduced if there is applied tension. The factor, k_s, is a multiplier that reduces the nominal slip resistance given by Equation J3-4 as a function of the applied tension load.

10. Bearing Strength at Bolt Holes

Provisions for bearing strength of pins differ from those for bearing strength of bolts; refer to Section J7.

Bearing strength values are provided as a measure of the strength of the material upon which a bolt bears, not as a protection to the fastener, which needs no such protection. Accordingly, the same bearing value applies to all joints assembled by bolts, regardless of fastener shear strength or the presence or absence of threads in the bearing area.

Material bearing strength may be limited either by bearing deformation of the hole or by tearout (a bolt-by-bolt block shear rupture) of the material upon which the bolt bears. Kim and Yura (1996) and Lewis and Zwerneman (1996) confirmed the bearing strength provisions for the bearing case wherein the nominal bearing strength R_n is equal to $CdtF_u$ and C is equal to 2.4, 3.0 or 2.0 depending upon hole type and/or acceptability of hole ovalization at ultimate load, as indicated in Section J3.10. However, this same research indicated the need for different bearing strength provisions when tearout failure would control. Appropriate equations for bearing strength as a function of clear distance L_c are therefore provided and this formulation is consistent with that in the RCSC *Specification* (RCSC, 2004).

Frank and Yura (1981) demonstrated that hole elongation greater than $^1/_4$ in. (6 mm) will generally begin to develop as the bearing force is increased beyond $2.4dtF_u$, especially if it is combined with high tensile stress on the net section, even though rupture does not occur. For a long-slotted hole with the slot perpendicular to the direction of force, the same is true for a bearing force greater than $2.0dtF_u$. An upper bound of $3.0dtF_u$ anticipates hole ovalization [deformation greater than $^1/_4$ in. (6 mm)] at maximum strength.

Additionally, to simplify and generalize such bearing strength calculations, the current provisions have been based upon a clear-distance formulation. Previous provisions utilized edge distances and bolt spacings measured to hole centerlines with adjustment factors to account for varying hole type and orientation, as well as minimum edge distance requirements.

11. Tension Fasteners

With any connection configuration where the fasteners transmit a tensile force to the HSS wall, a rational analysis must be used to determine the appropriate limit states. These may include a yield-line mechanism in the HSS wall and/or pull-out through the HSS wall, in addition to applicable limit states for the fasteners subject to tension.

J4. AFFECTED ELEMENTS OF MEMBERS AND CONNECTING ELEMENTS

Sections J4 and J5 of previous editions of the Specification have been combined in Section J4.

1. Strength of Elements in Tension

Tests have shown that yielding will occur on the gross section before the tensile capacity of the net section is reached if the ratio A_n/A_g is greater than or equal to

0.85 (Kulak and others, 1987). Since the length of connecting elements is small compared to the member length, inelastic deformation of the gross section is limited. Hence, the effective net area A_n of the connecting element is limited to $0.85A_g$ in recognition of the limited capacity for inelastic deformation, and to provide a reserve capacity.

2. Strength of Elements in Shear

In previous editions of the LRFD Specifications, the resistance factor for shear yielding had been 0.90, equivalent to a safety factor of 1.67. In ASD, the allowable shear yielding stress was $0.4F_y$, equivalent to a safety factor of 1.5. To make the LRFD approach in this Specification consistent with prior editions of the *ASD Specification*, the resistance and safety factors for shear yielding in this Specification are 1.0 and 1.5, respectively. The resulting increase in LRFD design strength of approximately 10 percent is justified by the long history of satisfactory performance of ASD use.

3. Block Shear Strength

Tests on coped beams indicated that a tearing failure mode (rupture) can occur along the perimeter of the bolt holes as shown in Figure C-J4.1 (Birkemoe and Gilmor, 1978). This block shear mode combines tensile failure on one plane and shear failure on a perpendicular plane. The failure path is defined by the centerlines of the bolt holes.

The block shear failure mode is not limited to coped ends of beams; other examples are shown in Figures C-J4.1 and C-J4.2. The block shear failure mode must also be checked around the periphery of welded connections.

This Specification has adopted a conservative model to predict block shear strength. The mode of failure in coped beam webs and angles is different than that of gusset plates because the shear resistance is present on only one plane, in which case there must be some rotation of the block of material that is providing the total resistance. Although tensile failure is observed through the net section

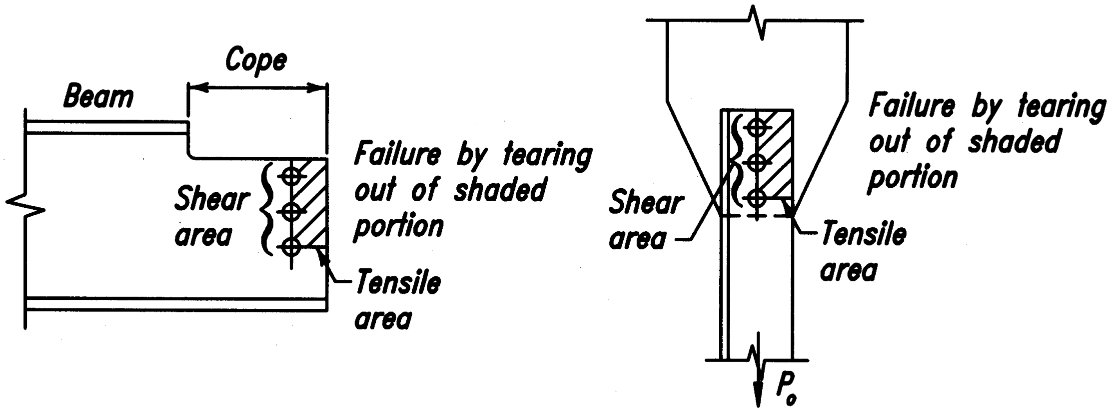
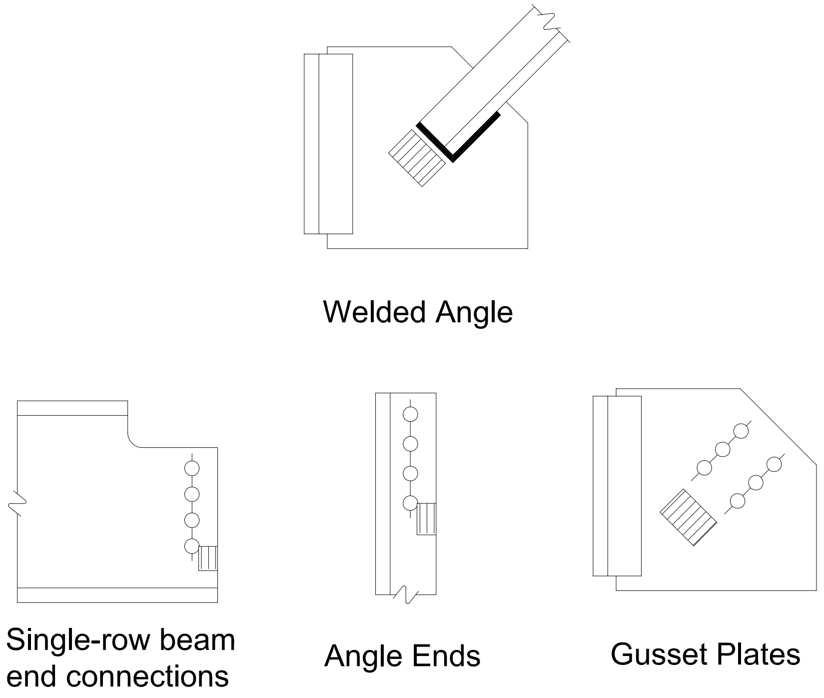


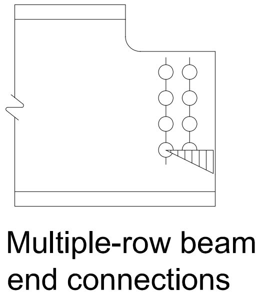
Fig. C-J4.1. Failure surface for block shear rupture limit state.

on the end plane, the distribution of tensile stresses is not always uniform (Ricles and Yura, 1983; Kulak and Grondin, 2001a). A reduction factor, U_{bs}, has been included in Equation J4-5 to approximate the non-uniform stress distribution on the tensile plane. The tensile stress distribution is non-uniform in the two row connection in Figure C-J4.2(b) because the rows of bolts nearest the beam end pick up most of the shear load.

Block shear is a rupture or tearing phenomenon, not a yielding limit state. However, gross yielding on the shear plane can occur when tearing on the tensile plane commences if $0.6F_uA_{nv}$ exceeds $0.6F_yA_{gv}$. Hence, Equation J4-5 limits the term $0.6F_yA_{gv}$ to not greater than $0.6F_uA_{nv}$. Equation J4-5 is consistent with the philosophy in Chapter D for tension members where the gross area is used for the limit state of yielding and the net area is used for the limit state of rupture.



(a) Cases for which U_{bs}= 1.0



(b) Case for which U_{bs}= 0.5

Fig. C-J4.2. Block shear tensile stress distributions.

4. Strength of Elements in Compression

To simplify connection calculations, the nominal strength of elements in compression when the element slenderness ratio is not greater than 25 is $F_y A_g$, which is a very slight increase over that obtained if the provisions of Chapter F are used. For more slender elements, the provisions of Chapter F apply.

J5. FILLERS

The practice of securing fillers by means of additional fasteners, so that they are, in effect, an integral part of a shear-connected component, is not required where a connection is designed for slip at member required strength levels. In such connections, the resistance to slip between the filler and either connected part is comparable to that which would exist between the connected parts if no filler were present.

Filler plates may be used in lap joints of welded connections that splice parts of different thickness, or where there may be an offset in the joint.

J6. SPLICES

The nominal strength of the smaller plate must be developed when groove-welded splices are used in plate girders and beams. For other connections it is sufficient to provide a connection to resist the required force at the joint.

J7. BEARING STRENGTH

In general, the bearing strength design of milled surfaces is governed by the limit state of bearing (local compressive yielding) at nominal loads, resulting in a stress of $0.9F_y$. Adequate safety is provided by post-yield strength as deformation increases. Tests on pin connections (Johnston, 1939) and on rockers (Wilson, 1934) have confirmed this behavior.

As used throughout the Specification, the terms “milled surface,” “milled” and “milling” are intended to include surfaces that have been accurately sawed or finished to a true plane by any suitable means.

J8. COLUMN BASES AND BEARING ON CONCRETE

The provisions of this section are identical to equivalent provisions in ACI 318 (ACI, 2002).

J9. ANCHOR RODS AND EMBEDMENTS

The term “anchor rod” is used for threaded rods embedded in concrete to anchor structural steel. The term “rod” is intended to clearly indicate that these are threaded rods, not structural bolts, and should be designed as threaded parts per Table J3.2 using the material specified in Section A3.4.

Generally, the largest tensile force for which anchor rods need to be designed is that produced by bending moment at the column base and augmented by any uplift caused by the overturning tendency of a building under lateral load.

Shear at the base of a column is seldom resisted by bearing of the column base plate against the anchor rods. Even considering the lowest conceivable slip coefficient, the friction due to the vertical load on a column is generally more than sufficient to result in the transfer by frictional resistance of any likely amount of shear from the column base to the foundation. The possible exception is at the base of braced frames and moment frames where larger shear forces may require that shear transfer be accomplished by embedding the column base or providing a shear key at the top of the foundation.

The anchor rod hole sizes listed in Tables C-J9.1 and C-J9.1M are recommended to accommodate the tolerance required for setting anchor rods cast in concrete. These larger hole sizes are not detrimental to the integrity of the supported structure when used with proper washers. The slightly conical hole that results from punching operations or thermal cutting is acceptable.

If plate washers are utilized to resolve horizontal shear, bending in the anchor rod must be considered in the design and the layout of anchor rods must accommodate plate washer clearances. In this case special attention must be given to weld clearances, accessibility, edge distances on plate washers, and the effect of the tolerances between the anchor rod and the edge of the hole.

It is important that the placement of anchor rods be coordinated with the placement and design of reinforcing steel in the foundations as well as the design and overall size of base plates. It is recommended that the anchorage device at the anchor rod bottom be as small as possible to avoid interference with the reinforcing steel in the foundation. A heavy-hex nut or forged head is adequate to develop the concrete shear cone. See DeWolf and Ricker (1990) for design of base plates and anchor rods along with ACI 318 (ACI, 2002) and ACI 349 (ACI, 2001) for embedment design. Also see OSHA *Safety and Health Regulations for Construction*, Standards—29 CFR 1926 Subpart R—Steel Erection (OSHA, 2001) for anchor rod design and construction requirements for erection safety.

J10. FLANGES AND WEBS WITH CONCENTRATED FORCES

This Specification separates flange and web strength requirements into distinct categories representing different limit states, namely, flange local bending (Section J10.1), web local yielding (Section J10.2), web crippling (Section J10.3), web *sidesway buckling* (Section J10.4), web compression buckling (Section J10.5), and web panel-zone shear (Section J10.6).

These limit state provisions are applied to two distinct types of concentrated forces normal to member flanges:

TABLE C-J9.1
Anchor Rod Hole Diameters, in.

Anchor Rod Diameter	Anchor Rod Hole Diameter
$^{1}/_{2}$	$1^{1}/_{16}$
$^{5}/_{8}$	$1^{3}/_{16}$
$^{3}/_{4}$	$1^{5}/_{16}$
$^{7}/_{8}$	$1^{9}/_{16}$
1	$1^{13}/_{16}$
$1^{1}/_{4}$	$2^{1}/_{16}$
$1^{1}/_{2}$	$2^{5}/_{16}$
$1^{3}/_{4}$	$2^{3}/_{4}$
≥ 2	$d_b + 1^{1}/_{4}$

TABLE C-J9.1M
Anchor Rod Hole Diameters, mm.

Anchor Rod Diameter	Anchor Rod Hole Diameter
18	32
22	36
24	42
27	48
30	51
33	54
36	60
39	63
42	74

Single concentrated forces may be tensile (such as those delivered by tension hangers) or compressive (such as those delivered by bearing plates at beam interior positions, reactions at beam ends, and other bearing connections). Flange local bending applies only for tensile forces, web local yielding applies to both tensile and compressive forces, and the remainder of these limit states apply only to compressive forces. Double concentrated forces, one tensile and one compressive, form a couple on the same side of the loaded member, such as that delivered to column flanges through welded and bolted moment connections.

Transverse stiffeners, also called continuity plates, and web doubler plates are only required when the demand (the transverse concentrated force) exceeds the available strength. It is often more economical to choose a heavier member than to provide such reinforcement (Carter, 1999; Troup, 1999). The demand may be determined as the largest flange force from the various load cases, although the demand may also be taken as the gross area of the attachment delivering the force multiplied by the specified minimum yield strength, F_y. Stiffeners and/or doublers and their attaching welds are sized for the difference between the demand and the applicable limit state strength. Requirements for stiffeners are provided in Sections J10.7 and J10.8 and requirements for doublers are provided in Section J10.9.

1. Flange Local Bending

Where a tensile force is applied through a plate welded across a flange, that flange must be sufficiently rigid to prevent deformation of the flange and the corresponding high stress concentration in the weld in line with the web.

The effective column flange length for local flange bending is $12t_f$ (Graham, Sherbourne, Khabbaz, and Jensen, 1960). Thus, it is assumed that yield lines form in the flange at $6t_f$ in each direction from the point of the applied concentrated force. To develop the fixed edge consistent with the assumptions of this model, an additional $4t_f$, and therefore a total of $10t_f$, is required for the full flange-bending strength given by Equation J10-1. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the applied concentrated force is less than $10t_f$ from the member end.

The strength given by Equation J10-1 was originally developed for moment connections but also applies to single concentrated forces such as tension hangers consisting of a plate welded to the bottom flange of a beam and transverse to the beam web. In the original tests, the strength given by Equation J10-1 was intended to provide a lower bound to the force required for weld fracture, which was aggravated by the uneven stress and strain demand on the weld caused by the flange deformation (Graham, Sherbourne, and Khabbaz, 1959).

Recent tests on welds with minimum Charpy V-Notch (CVN) toughness requirements show that weld fracture is no longer the failure mode when the strength given by Equation J10-1 is exceeded. Rather, it was found that the strength given by Equation J10-1 is consistently less than the force required to separate the flanges in typical column sections by $^1/_4$ in. (6 mm) (Hajjar, Dexter, Ojard, Ye, and Cotton, 2003; Prochnow, Ye, Dexter, Hajjar, and Cotton, 2000). This amount of flange deformation is on the order of the tolerances in ASTM A6, and it is believed that if the flange deformation exceeded this level it could be detrimental to other aspects of the performance of the member, such as flange local buckling. Although this deformation could also occur under compressive normal forces, it is customary that flange local bending is checked only for tensile forces (because the original concern was weld fracture). Therefore it is not required to check flange local bending for compressive forces.

The provision in Section J10.1 is not applicable to moment end-plate and tee-stub type connections. For these connections, see Carter (1999) or the AISC *Manual of Steel Construction* (AISC, 2005a).

2. Web Local Yielding

The web local yielding provisions (Equations J10-2 and J10-3) apply to both compressive and tensile forces of bearing and moment connections. These provisions are intended to limit the extent of yielding in the web of a member into which a force is being transmitted. The provisions are based on tests on two-sided directly welded girder-to-column connections (cruciform tests) (Sherbourne and Jensen, 1957) and were derived by considering a stress zone that spreads out with a slope of 2:1. Graham and others (1960) report pull-plate tests and suggest that a 2.5:1 stress gradient would be more appropriate.

Recent tests confirm that the provisions given by Equations J10-2 and J10-3 are slightly conservative and that the yielding is confined to a length consistent with the slope of 2.5:1 (Hajjar and others, 2003; Prochnow and others, 2000).

3. Web Crippling

The web crippling provisions (Equations J10-4 and J10-5) apply only to compressive forces. Originally, the term “web crippling” was used to characterize phenomena now called local web yielding, which was then thought to also adequately predict web crippling. The first edition of the AISC *LRFD Specification* (AISC, 1986) was the first AISC Specification to distinguish between local web yielding and local web crippling. Web crippling was defined as crumpling of the web into buckled waves directly beneath the load, occurring in more slender webs, whereas web local yielding is yielding of that same area, occurring in stockier webs.

Equations J10-4 and J10-5 are based on research reported in Roberts (1981). The increase in Equation J10-5b for $N/d > 0.2$ was developed after additional testing to better represent the effect of longer bearing lengths at ends of members (Elgaaly and Salkar, 1991). All tests were conducted on bare steel beams without the expected beneficial contributions of any connection or floor attachments. Thus, the resulting provisions are considered conservative for such applications. Kaczinski, Schneider, Dexter, and Lu (1994) reported tests on cellular box beams with slender webs and confirmed that these provisions are appropriate in this type of structure as well.

The equations were developed for bearing connections but are also generally applicable to moment connections.

The web crippling phenomenon has been observed to occur in the web adjacent to the loaded flange. For this reason, a half-depth stiffener (or stiffeners) or a half-depth doubler plate is needed to eliminate this limit state.

4. Web Sidesway Buckling

The web *sidesway buckling* provisions (Equations J10-6 and J10-7) apply only to compressive forces in bearing connections and do not apply to moment connections. The web *sidesway buckling* provisions were developed after observing several unexpected failures in tested beams (Summers and Yura, 1982; Elgaaly, 1983). In those tests the compression flanges were braced at the concentrated load, the web was subjected to compression from a concentrated load applied to the flange and the tension flange buckled (see Figure C-J10.1).

Web *sidesway buckling* will not occur in the following cases:

(a) For flanges restrained against rotation (such as when connected to a slab), when

$$\frac{h/t_w}{l/b_f} > 2.3 \qquad \text{(C-J10-1)}$$

(b) For flanges *not* restrained against rotation, when

$$\frac{h/t_w}{l/b_f} > 1.7 \qquad \text{(C-J10-2)}$$

where l is as shown in Figure C-J10.2.

Web *sidesway buckling* can be prevented by the proper design of lateral bracing or stiffeners at the load point. It is suggested that local bracing at both flanges be designed for 1 percent of the concentrated force applied at that point. If stiffeners are used, they must extend from the load point through at least one-half the beam or girder depth. In addition, the pair of stiffeners must be designed to carry the full load. If flange rotation is permitted at the loaded flange, neither stiffeners nor doubler plates are effective.

5. Web Compression Buckling

The web compression buckling provision (Equation J10-8) applies only when there are compressive forces on both flanges of a member at the same cross section, such as might occur at the bottom flange of two back-to-back moment connections under gravity loads. Under these conditions, the member web must have its slenderness ratio limited to avoid the possibility of buckling. Equation J10-8 is applicable to a pair of moment connections, and to other pairs of compressive forces applied at both flanges of a member, for which N/d is approximately less than 1. When N/d is not small, the member web should be designed as a compression member in accordance with Chapter E.

Equation J10-8 is predicated on an interior member loading condition. In the absence of applicable research, a 50 percent reduction has been introduced for cases wherein the compressive forces are close to the member end.

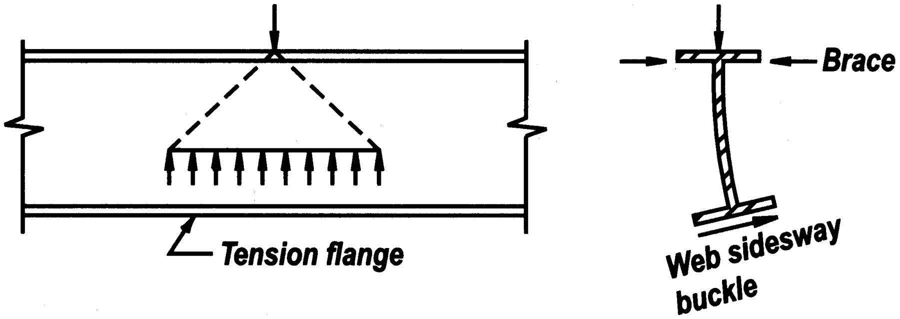


Fig. C-J10.1. Web sidesway buckling.

6. Web Panel-Zone Shear

Column web shear stresses may be significant within the boundaries of the rigid connection of two or more members with their webs in a common plane. Such webs must be reinforced when the required force ΣF_u for LRFD or ΣF for ASD along plane A-A in Figure C-J10.3 exceeds the column web available strength ϕR_v or R_v / Ω, respectively, where

for LRFD

$$\Sigma F_u = \frac{M_{u1}}{d_{m1}} + \frac{M_{u2}}{d_{m2}} - V_u \qquad \text{(C-J10-3a)}$$

and

$M_{u1} = M_{u1L} + M_{u1G}$ = sum of the moments due to the factored lateral loads, M_{u1L}, and the moments due to factored gravity loads, M_{u1G}, on the windward side of the connection, kip-in. (N-mm)

$M_{u2} = M_{u2L} - M_{u2G}$ = difference between the moments due to the factored lateral loads M_{u2L} and the moments due to factored gravity loads, M_{u2G}, on the windward side of the connection, kip-in. (N-mm)

for ASD

$$\Sigma F = \frac{M_{a1}}{d_{m1}} + \frac{M_{a2}}{d_{m2}} - V \qquad \text{(C-J10-3b)}$$

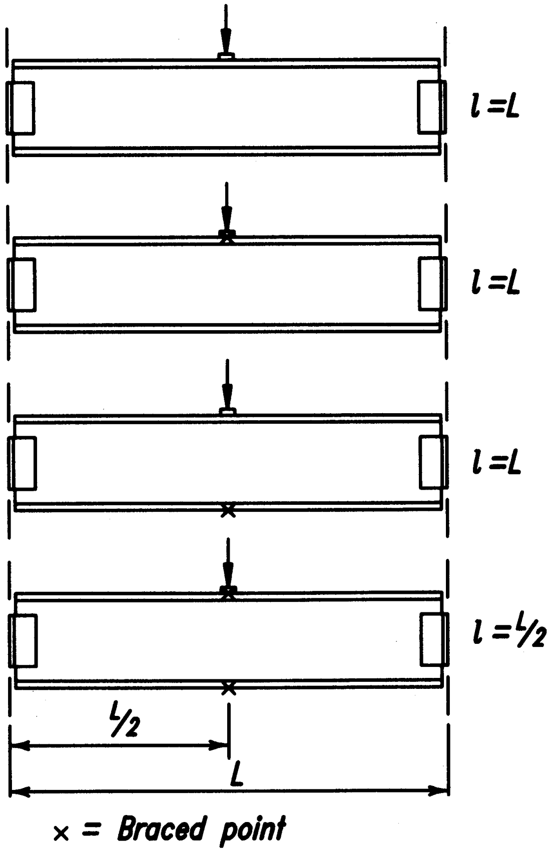


Fig. C-J10.2. Unbraced flange length for web sidesway buckling.

and

$M_{a1} = M_{a1L} + M_{a1G}$ = sum of the moments due to the nominal lateral loads, M_{a1L}, and the moments due to nominal gravity loads, M_{aG}, on the windward side of the connection, kip-in. (N-mm)

$M_{a2} = M_{a2L} + M_{a2G}$ = difference between the moments due to the nominal lateral loads, M_{a2L}, and the moments due to nominal gravity loads, M_{a2G}, on the windward side of the connection, kip-in. (N-mm)

d_{m1}, d_{m2} = distance between flange forces in the moment connection, in. (mm)

Historically (and conservatively), 0.95 times the beam depth has been used for d_m.

If, for LRFD $\Sigma F_u \leq \phi R_v$, or for ASD $\Sigma F \leq R_v / \Omega$, no reinforcement is necessary, in other words, $t_{req} \leq t_w$, where t_w is the column web thickness.

Equations J10-9 and J10-10 limit panel-zone behavior to the elastic range. While such connection panels possess large reserve capacity beyond initial general shear yielding, the corresponding inelastic joint deformations may adversely affect the strength and stability of the frame or story (Fielding and Huang, 1971; Fielding and Chen, 1973). Panel-zone shear yielding affects the overall frame stiffness and, therefore, the resulting second-order effects may be significant. The shear/axial interaction expression of Equation J10-10, as shown in Figure C-J10.4, provides elastic panel behavior.

If adequate connection ductility is provided and the frame analysis considers the inelastic panel-zone deformations, then the additional inelastic shear strength is recognized in Equations J10-11 and J10-12 by the factor

$$\left(1 + \frac{3b_{cf}t_{cf}^2}{d_b d_c t_w}\right)$$

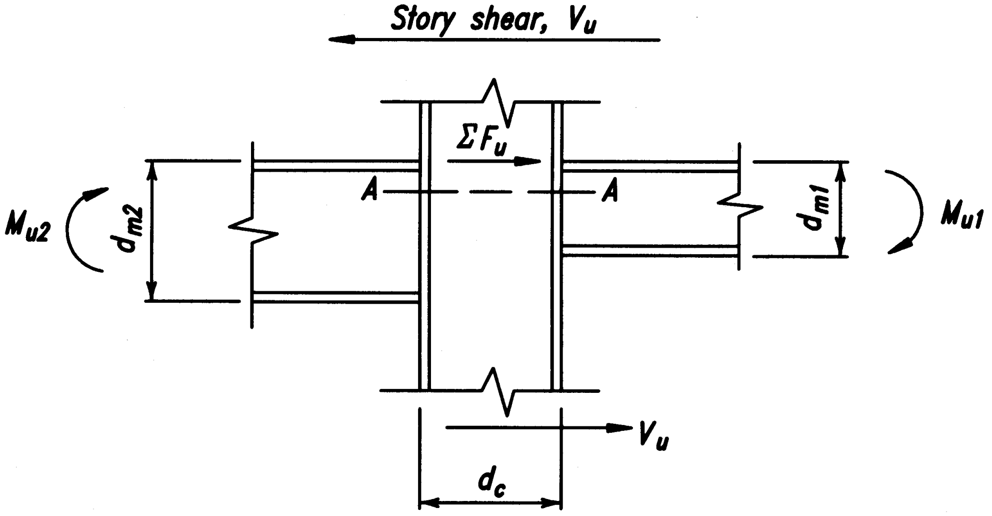


Fig. C-J10.3. LRFD forces in panel zone (ASD forces are similar).

This inelastic shear strength has been most often utilized for the design of frames in high seismic design and should be used when the panel zone is designed to develop the strength of the members from which it is formed.

The shear/axial interaction expression incorporated in Equation J10-12 (see Figure C-J10.5) recognizes that when the panel-zone web has completely yielded in shear, the axial column load is carried in the flanges.

7. Unframed Ends of Beams and Girders

Full-depth stiffeners are required at unframed ends of beams and girders not otherwise restrained to avoid twisting about their longitudinal axes.

8. Additional Stiffener Requirements for Concentrated Forces

See Carter (1999), Troup (1999), and Murray and Sumner (2004) for guidelines on column stiffener design.

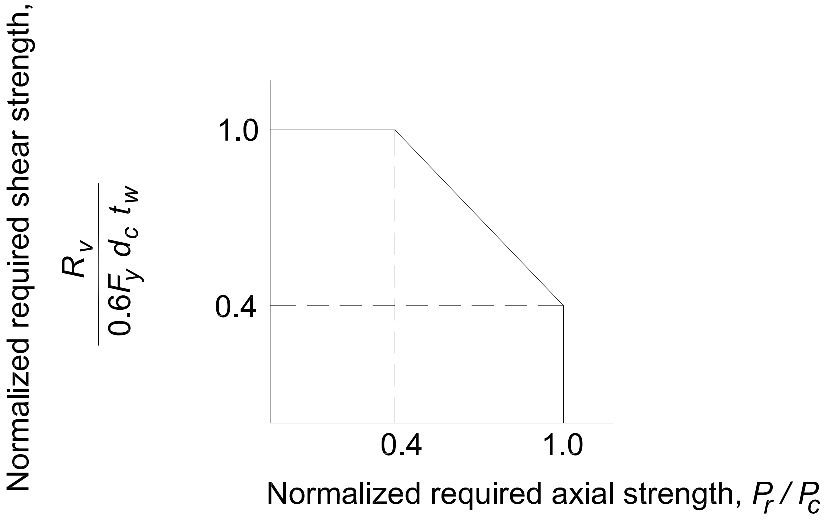


Fig. C-J10.4. Interaction of shear and axial force—elastic.

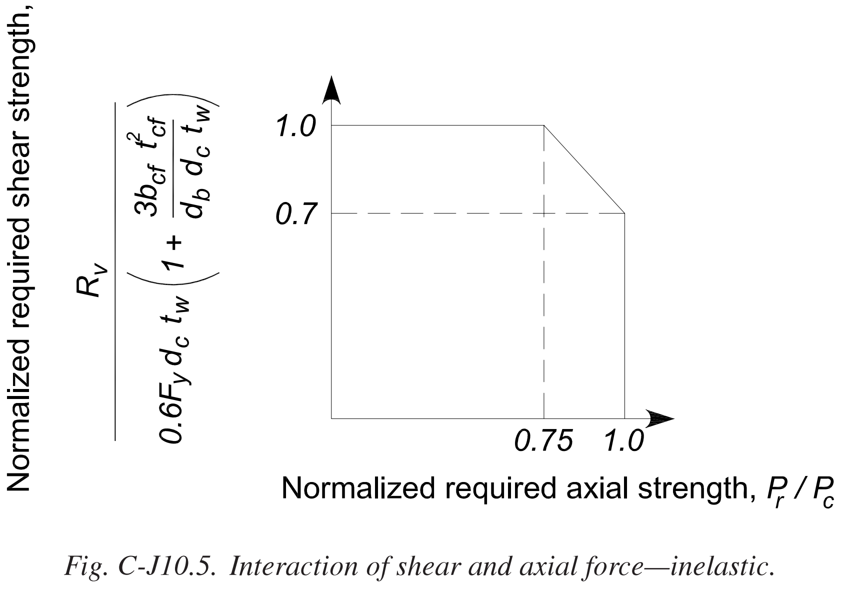


Fig. C-J10.5. Interaction of shear and axial force—inelastic.

For rotary straightened W-shapes, an area of reduced notch toughness is sometimes found in a limited region of the web immediately adjacent to the flange, referred to as the “k-area,” as illustrated in Figure C-J10.6 (Kaufmann, Metrovich, Pense, and Fisher, 2001). Following the 1994 Northridge Earthquake, there was a tendency to specify thicker continuity plates that were groove welded to the web and flange and thicker doubler plates that were often groove welded in the gap between the doubler plate and the flanges. These welds were highly restrained and may have caused cracking during fabrication in some cases (Tide, 1999).

AISC (1997a) recommended that the welds for continuity plates should terminate away from the k-area, which is defined as the “region extending from approximately the midpoint of the radius of the fillet into the web approximately 1 to 1½ in. (25 to 38 mm) beyond the point of tangency between the fillet and web.”

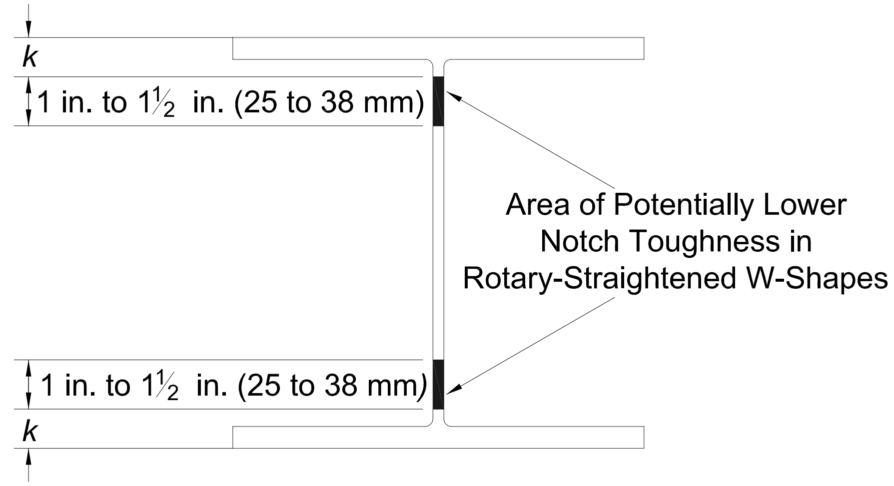


Fig. C-J10.6. Representative “k-area” of a wide-flange shape.

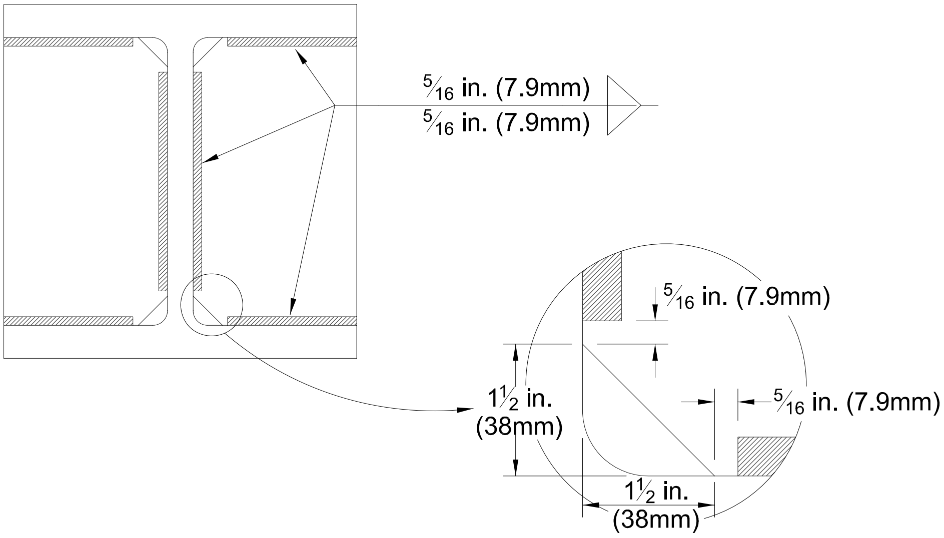


Fig. C-J10.7. Recommended placement of stiffener fillet welds to avoid contact with “k-area.”

Recent pull-plate testing (Dexter and Melendrez, 2000; Prochnow and others, 2000; Hajjar and others, 2003) and full-scale beam-column joint testing (Bjorhovde, Goland, and Benac, 1999; Dexter, Hajjar, Prochnow, Graeser, Galambos, and Cotton, 2001; Lee, Cotton, Dexter, Hajjar, Ye, and Ojard, 2002) has shown that this problem can be avoided if the column stiffeners are fillet welded to both the web and the flange, the corner is clipped at least $1^{1}/_{2}$ in. (38 mm), and the fillet welds are stopped short by a weld leg length from the edges of the cutout, as shown in Figure C-J10.7. These tests also show that groove welding the stiffeners to the flanges or the web is unnecessary, and that the fillet welds performed well with no problems. If there is concern regarding the development of the stiffeners using fillet welds, the corner clip can be made so that the dimension along the flange is $^{3}/_{4}$ in. (20 mm) and the dimension along the web is $1^{1}/_{2}$ in. (38 mm).

Recent tests have also shown the viability of fillet welding doubler plates to the flanges, as shown in Figure C-J10.8 (Prochnow and others, 2000; Dexter and others, 2001; Lee and others, 2002; Hajjar and others, 2003). It was found that it

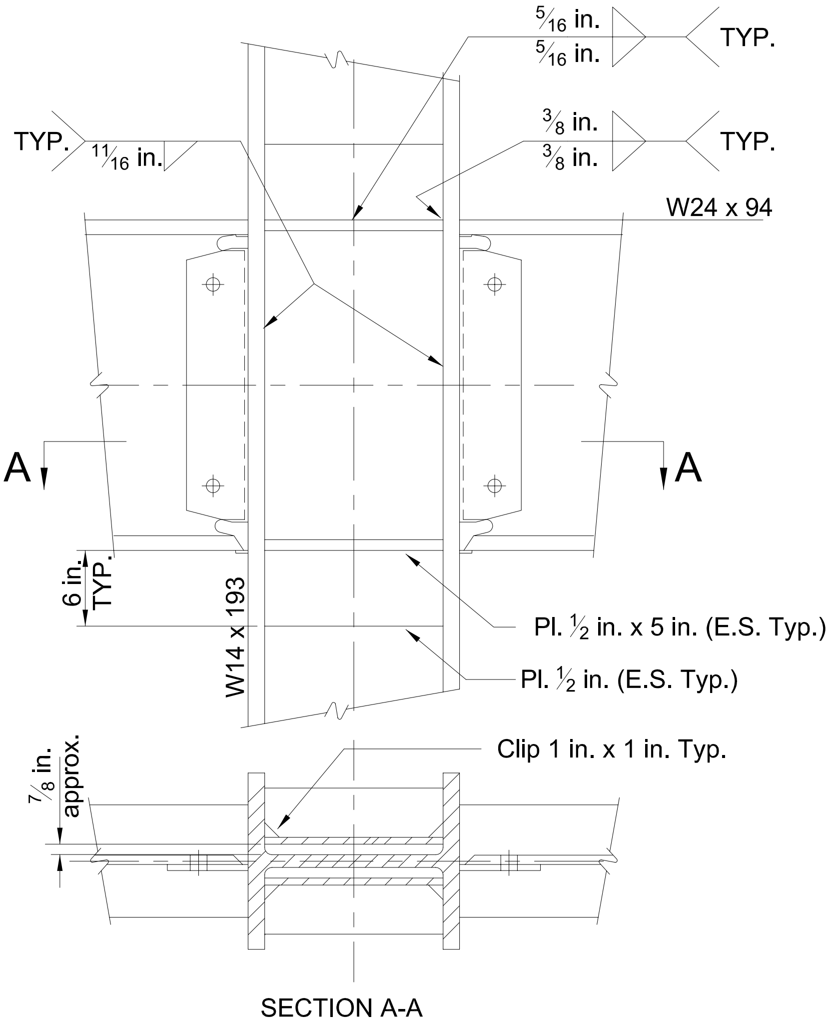


Fig. C-J10.8. Example of fillet welded doubler plate and stiffener details.

is not necessary to groove weld the doubler plates and that they do not need to be in contact with the column web to be fully effective.

9. Additional Doubler Plate Requirements for Concentrated Forces

When required, doubler plates are to be designed using the appropriate limit state requirements for the type of loading. The sum of the strengths of the member element and the double plate(s) must exceed the required strength and the doubler plate must be welded to the member element.

CHAPTER K

DESIGN OF HSS AND BOX MEMBER CONNECTIONS

Chapter K addresses the strength of HSS and box member welded connections. The provisions are based upon failure modes that have been reported in international research on HSS, much of which has been sponsored and synthesized by CIDECT (International Committee for the Development and Study of Tubular Construction) since the 1960s. This work has also received critical review by the International Institute of Welding (IIW) Subcommission XV-E on "Welded Joints in Tubular Structures." The HSS connection design recommendations are generally in accord with the last edition of the design recommendations by this Subcommission (IIW, 1989). Some minor modifications to the IIW recommended provisions for some limit states have been made by the adoption of the formulations for the same limit states elsewhere in this Specification. The IIW connection design recommendations referred to above have also been implemented and supplemented in later design guides by CIDECT (Wardenier, Kurobane, Packer, Dutta, and Yeomans, 1991; Packer, Wardenier, Kurobane, Dutta, and Yeomans, 1992), in the design guide by the Canadian Institute of Steel Construction (Packer and Henderson, 1997) and in Eurocode 3 (2002). Parts of these IIW design recommendations are also incorporated in AWS (2004). A large amount of research data generated by CIDECT research programs up to the mid-1980s is summarized in CIDECT Monograph No. 6 (Giddings and Wardenier, 1986). Further information on CIDECT publications and reports can be obtained from their website: *www.cidect.com.*

The scopes of Sections K2 and K3 note that the centerlines of the branch member(s) and the chord members must lie in a single plane. For other configurations, such as multiplanar connections, connections with partially or fully flattened branch member ends, double chord connections, connections with a branch member that is offset so that its centerline does not intersect with the centerline of the chord or connections with round branch members joined to a square or rectangular chord member, the provisions of IIW (1989), CIDECT, Wardenier and others (1991), Packer and others (1992), CISC, Packer and Henderson (1997), Marshall (1992), AWS (2004), or other verified design guidance or tests can be used.

K1. CONCENTRATED FORCES ON HSS

1. Definitions of Parameters

Some of the notation used in Chapter K is illustrated in Figure C-K1.1.

2. Limits of Applicability

The limits of applicability in Section K1.2 stem primarily from limitations on tests conducted to date.

3. Concentrated Force Distributed Transversely

Sections K1.3 and K1.4, although pertaining to all concentrated forces on HSS, are particularly oriented towards plate-to-HSS welded connections and this application is displayed in tabular form in Table C-K1.1 (a) and (b). In addition to the design provisions in the Specification, Table C-K1.1(b) also gives flexural strengths for some plate-to-round HSS connections. Most of the equations (after application of appropriate resistance factors for LRFD) conform to CIDECT Design Guides 1 and 3 (Wardenier and others, 1991; Packer and others, 1992) with updates in accordance with CIDECT Design Guide 9 (Kurobane, Packer, Wardenier, and Yeomans, 2004). The latter includes revisions for longitudinal plate-to-rectangular HSS connections (Equation K1-9) based on extensive experimental and numerical studies reported in Kosteski and Packer (2003). The provisions for the limit state of sidewall crippling of rectangular HSS, Equations K1-5 and K1-6, conform to web crippling expressions elsewhere in this Specification, and not to CIDECT or IIW recommendations. If a longitudinal plate-to-rectangular HSS connection is made by passing the plate through a slot in the HSS and then welding the plate to both the front and back HSS faces to form a "through-plate connection," the nominal strength can be taken as twice that given by Equation K1-9 (Kosteski and Packer, 2003).

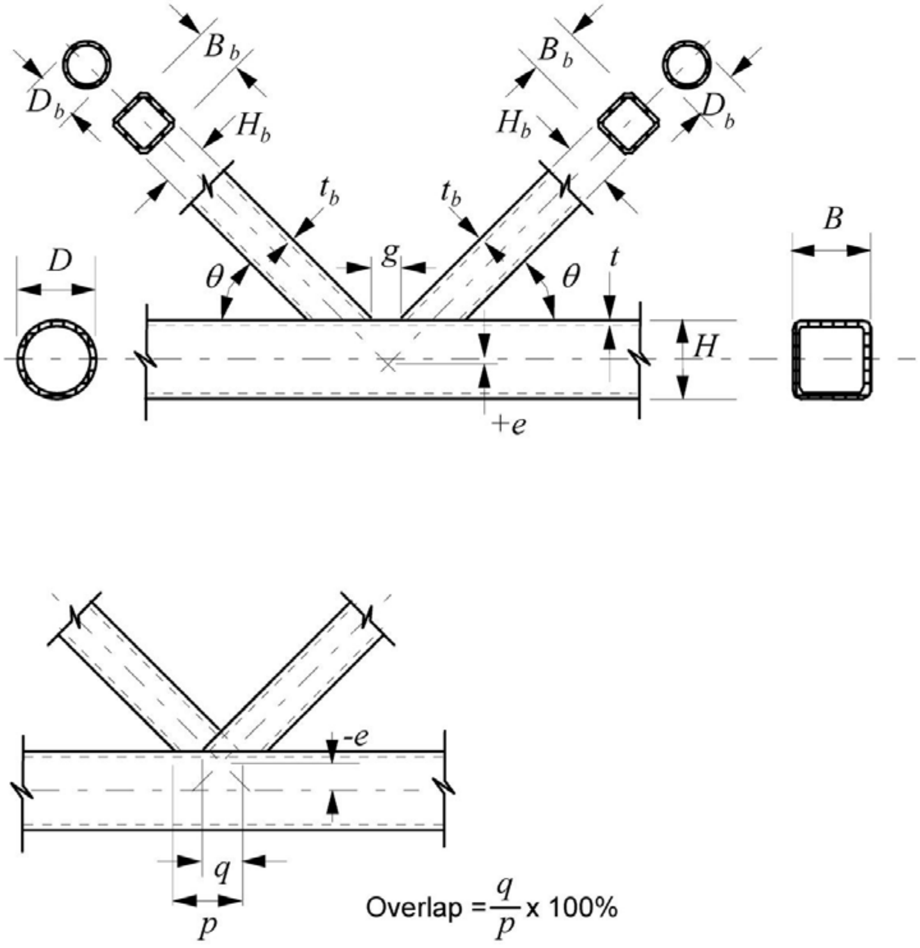


Fig. C-K1.1. Common notation for HSS connections.

The equations given for transverse plate-to-HSS connections can also be adapted for wide-flange beam-to-HSS PR moment connections, by treating the beam flanges as a pair of transverse plates and ignoring the beam web. For such wide-flange beam connections, the beam moment is thus produced by a force couple in the beam flanges. The connection flexural strength is then given by the plate-to-HSS connection strength multiplied by the distance between the beam flange centers. In Table C-K1.1(a) there is no check for the limit state of chord wall plastification for transverse plate-to-rectangular HSS connections, because this will not govern the design in practical cases. However, if there is a major compression load in the HSS, such as when it is used as a column, one should be aware that this compression load in the main member has a negative influence on the yield line plastification failure mode of the connecting chord wall (via a Q_f factor). In such a case, the designer can utilize guidance in CIDECT Design Guide No. 9 (Kurobane and others, 2004).

4. Concentrated Force Distributed Longitudinally at the Center of the HSS Diameter or Width, and Acting Perpendicular to the HSS Axis

See commentary for Section K1.3.

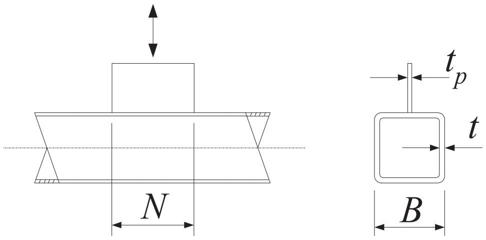
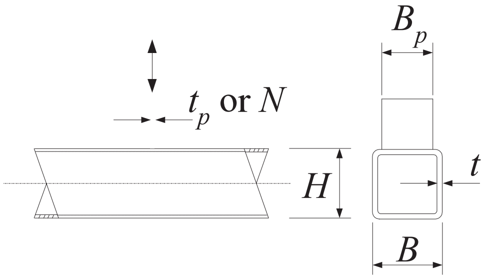
5. Concentrated Force Distributed Longitudinally at the Center of the HSS Width, and Acting Parallel to the HSS Axis

Section K1.5 applies to longitudinal plate connections loaded in shear. These recommendations are based on Sherman and Ales (1991), Sherman (1995a) and Sherman (1996) that investigated a large number of simple framing connections between wide-flange beams and rectangular HSS columns, in which the load transferred was predominantly shear. A review of costs also showed that single-plate and single-angle connections were the most economical, with double-angle and fillet-welded tee connections being more expensive. Through-plate and flare-bevel welded tee connections were among the most expensive (Sherman, 1995a). Over a wide range of connections tested, only one limit state was identified for the rectangular HSS column: punching shear failure related to end rotation of the beam, when a thick shear plate was joined to a relatively thin-walled HSS. Compliance with the inequality given by K1-10 precludes this HSS failure mode. This design rule is valid providing the HSS wall is not classified as a *slender element*. An extrapolation of inequality K1-10 has also been made for round HSS columns, subject to the round HSS cross-section not being classified as a *slender element*.

6. Concentrated Axial Force on the End of a Rectangular HSS with a Cap Plate

In Section K1.6, two limit states are given for the strength of a square or rectangular HSS wall with load transferred through a cap plate (or the flange of a T-stub), as shown in Figure C-K1.2. In general, the rectangular HSS could have dimensions of $B \times H$, but the illustration shows the bearing length (or width), N, oriented for lateral load dispersion into the wall of dimension B. A conservative distribution

TABLE C-K1.1 (a)
Nominal Strengths of Plate-to-Rectangular HSS Connections

Connection Type	**Connection Nominal Strength**
Longitudinal plate 	$\beta \le 0.85$ Basis: chord wall plastification $R_n = \dfrac{F_y t^2}{1-\dfrac{t_p}{B}}\left(\dfrac{2N}{B}+4\sqrt{1-\dfrac{t_p}{B}}\,Q_f\right)$
Transverse plate 	$\beta \approx 1.0$ Basis: HSS side wall strength Tension and compression: $R_n = 2F_yt[5k+N]$ Compression in T-connections: $R_n = 1.6t^2\left[1+\dfrac{3N}{H-3t}\right]\sqrt{EF_y}\,Q_f$ Compression in cross-connections: $R_n = \dfrac{48t^3}{H-3t}\sqrt{EF_y}\,Q_f$
	$0.85 \le \beta \le 1-2t/B$ Basis: punching shear failure
where $\beta = \dfrac{B_p}{B}$	$R_n = 0.6F_yt[2t_p+2B_{ep}]$
	All β Basis: uneven load distribution
	$R_n = \dfrac{10}{B/t}F_ytB_p \le F_{yp}t_pB_p$

Functions and Range of Validity

$\dfrac{B}{t} \le 35$ for the loaded HSS wall in transverse connections and

≤ 40 for longitudinal connections

$0.25 < \dfrac{B_p}{B} \le 1.0$ for transverse connections

$B_{ep} = \dfrac{10B_p}{B/t}$ but $\le B_p$

$k =$ outside corner radius of HSS $\ge 1.5t$

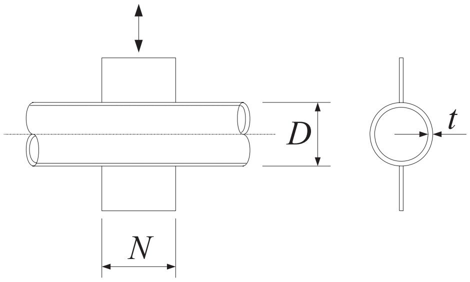
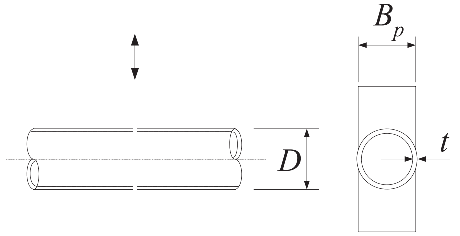
$Q_f = 1.0$ (chord in tension, for transverse connections)

$Q_f = 1.3 - 0.4\dfrac{U}{\beta}$ but ≤ 1.0 (chord in compression, for transverse connections)

$Q_f = \sqrt{1-U^2}$ (for longitudinal connections)

slope can be assumed as 2.5:1 from each face of the tee web (Wardenier and others, 1991; Kitipornchai and Traves, 1989), which produces a dispersed load width of $(5t_p + N)$. If this is less than B, only the two side walls of dimension B are effective in resisting the load, and even they will both be only partially effective. If $(5t_p + N) \ge B$, all four walls of the rectangular HSS will be engaged, and all

TABLE C-K1.1 (b)
Nominal Strengths of Plate-to-Round HSS Connections

	Connection Nominal Strength		
Connection Type	**Axial Force**	**Bending in Plane**	**Bending out of Plane**
Longitudinal plate 	Chord plastification: $R_n = 5.5 F_y t^2 \left(1 + 0.25 \frac{N}{D}\right) Q_f$	$M_n = N R_n$	—
Transverse plate 	$R_n = F_y t^2 \left[\frac{5.5}{1 - 0.81 \frac{B_p}{D}} \right] Q_f$	—	$M_n = 0.5 B_p R_n$

Functions and Range of Validity

$\frac{D}{t} \le 50$ for T-connections and ≤ 40 for cross-connections

$0.2 < \frac{B_p}{D} \le 1.0$ for transverse connections

$Q_f = 1.0$ (chord in tension)

$Q_f = 1.0 - 0.3U(1 + U)$ but ≤ 1.0 (chord in compression)

will be fully effective; however, the cap plate (or T-stub flange) must be sufficiently thick for this to happen. In Equations K1-11 and K1-12 the size of any weld legs has been conservatively ignored. If the weld leg size is known, it is acceptable to assume load dispersion from the toes of the welds. The same load dispersion model as shown in Figure C-K1.2 can also be applied to round HSS-to-cap plate connections.

K2. HSS-TO-HSS TRUSS CONNECTIONS

The classification of HSS truss-type connections as K- (which includes N-), Y- (which includes T-), or cross- (also known as X-) connections is based on the method of force transfer in the connection, not on the physical appearance of the connection. Examples of such classification are shown in Figure C-K2.1.

As noted in Section K2, when branch members transmit part of their load as K-connections and part of their load as T-, Y-, or cross-connections, the adequacy of each branch is determined by linear interaction of the proportion of the branch load involved in each type of load transfer. One K-connection, shown in Figure C-K2.1(b), illustrates that the branch force components normal to the chord member may differ by as much as 20 percent and still be deemed to exhibit K-connection behavior. This is to accommodate slight variations in branch member forces along a typical truss, caused by a series of panel point loads. The N-connection in Figure C-K2.1(c), however, has a ratio of branch force components normal to the chord member of 2:1. In this case, the connection is analyzed as both a "pure" K-connection (with balanced branch forces) and a cross- (or X-) connection (because the remainder of the diagonal branch load is being transferred through the connection), as shown in Figure C-K2.2. For the diagonal tension branch in that connection, the following check is also made:

$$\begin{gathered}(0.5P\sin\theta/K\text{-connection available strength})\\ +(0.5P\sin\theta/\text{cross-connection available strength})\le 1.0\end{gathered}$$

If the gap size in a gapped K- (or N-) connection [for example, Figure C-K2.1(a)] becomes large and exceeds the value permitted by the eccentricity limit, the "K-connection" should be treated as two independent Y-connections. In cross-connections, such as Figure C-K2.1(e), where the branches are close together or overlapping, the combined "footprint" of the two branches can be taken as the loaded area on the chord member. In K-connections such as Figure C-K2.1(d), where a branch has very little or no loading, the connection can be treated as a Y-connection, as shown.

The design of welded HSS connections is based on potential limit states that may arise for a particular connection geometry and loading, which in turn represent possible failure modes that may occur within prescribed limits of applicability.

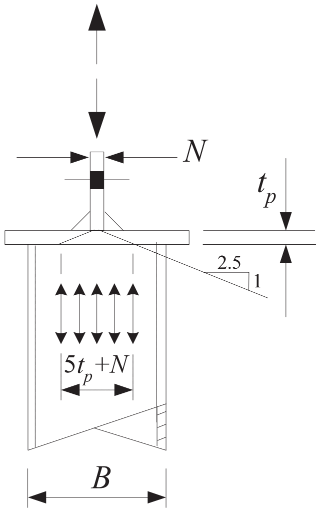


Fig. C-K1.2. Load dispersion from a concentrated force through a cap plate.

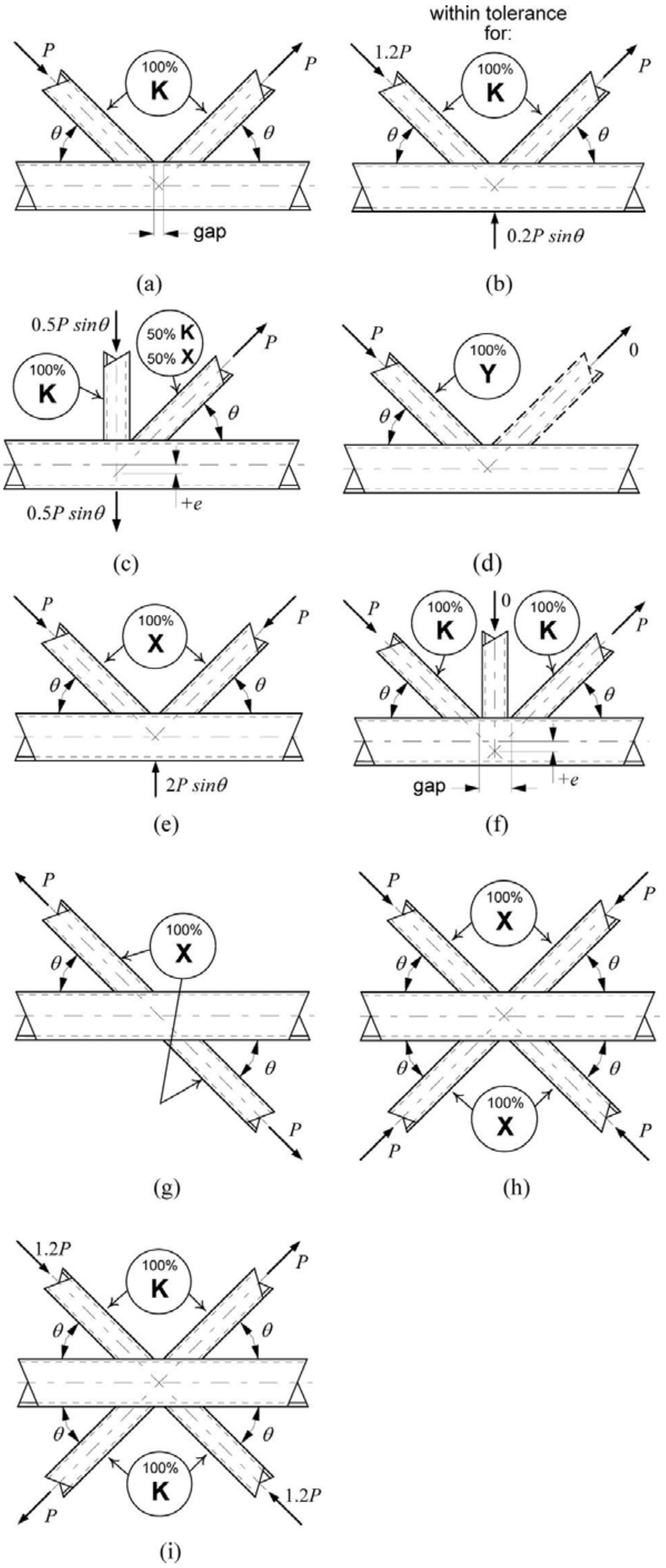


Fig. C-K2.1. Examples of HSS connection classification.

Some typical failure modes for truss-type connections, shown for rectangular HSS, are given in Figure C-K2.3.

1. Definitions of Parameters

Some parameters are defined in Figure C-K1.1.

2. Criteria for Round HSS

The limits of validity in Section K2.2a generally represent the parameter range over which the equations have been verified in experiments. The following limitations bear explanation:

(2) The minimum branch angle is a practical limit for good fabrication. Smaller branch angles are possible, but prior agreement with the fabricator should be made.
(5) The wall slenderness limit for the compression branch is a restriction so that connection strength is not reduced by branch local buckling.
(6) The minimum width ratio limit for gapped K-connections has been added in this Specification as a precaution, because Packer (2004) showed that for width ratios less than 0.4, Equation K2-6 may be potentially unconservative when evaluated against proposed equations for the design of such connections by the American Petroleum Institute (API, 1993).
(7) The restriction on the minimum gap size is only stated so that adequate space is available to enable welding at the toes of the branches to be satisfactorily performed.
(8) The restriction on the minimum overlap is applied so that there is an adequate interconnection of the branches, to enable effective shear transfer from one branch to the other.

The provisions given in Sections K2.2b and K2.2c are generally based, with the exception of the punching shear provision, on semi-empirical “characteristic strength” expressions, which have a confidence of 95 percent, taking into account the variation in experimental test results as well as typical variations in mechanical and geometric properties. These “characteristic strength” expressions are then multiplied by resistance factors for LRFD or divided by safety factors for ASD to further allow for the relevant failure mode. In the case of the chord plastification failure mode a ϕ factor of 0.9 or Ω factor of 1.67 is applied, whereas in the case

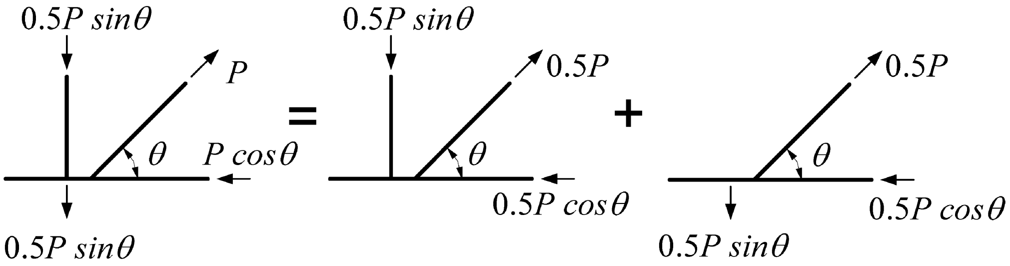
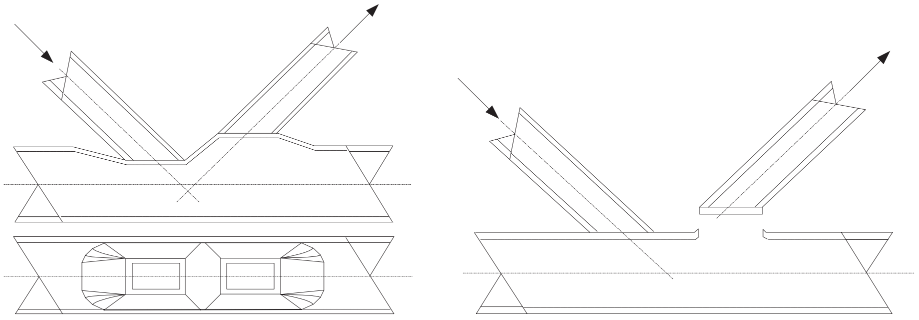
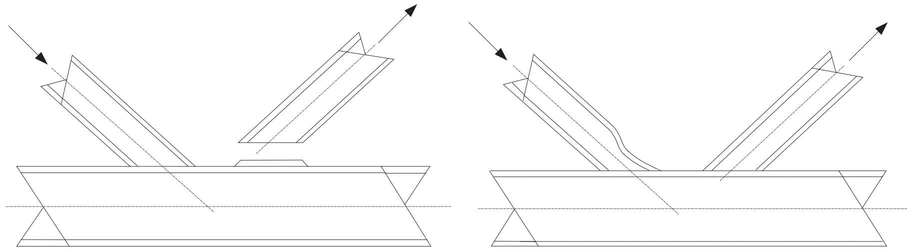


Fig. C-K2.2. Checking of K-connection with imbalanced branch member loads.

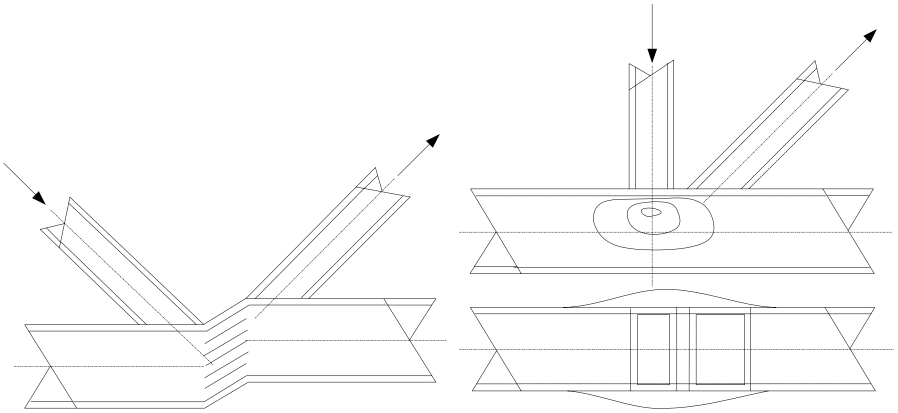
of punching shear a ϕ factor of 0.95 or a Ω factor of 1.58 is applied. The latter ϕ factor is 1.0 (equivalent to Ω of 1.50) in many recommendations or specifications [for example, IIW (1989), Packer and Henderson (1997), and Wardenier and others (1991)] to reflect the large degree of reserve strength beyond the analytical nominal strength expression, which is itself based on the shear yield (rather than ultimate) strength of the material. In this Specification, however, a ϕ factor of 0.95 or Ω factor of 1.58 is applied to maintain consistency with the factors for similar failure modes in Section K2.3. The shear failure resistance has also been taken as $0.95(0.6F_y) = 0.57F_y$, and elsewhere in Sections K2 and K3 as well, whereas



(a) Chord plastification *(b) Punching shear failure of the chord*



(c) Uneven load distribution, in the tension branch *(d) Uneven load distribution, in the compression branch*



(e) Shear yielding of the chord, in the gap *(f) Chord sidewall failure*

Fig. C-K2.3. Typical limit states for HSS-to-HSS truss connections.

IIW (1989) uses a von Mises shear yield resistance formulation of $1.0(F_y/\sqrt{3}) = 0.58F_y$. One should note that if the ultimate stress, F_u, were adopted as a basis for a punching shear rupture criterion, the accompanying ϕ would be 0.75 and Ω would be 2.0, as elsewhere in this Specification. Then, $0.75(0.6\ F_u) = 0.45\ F_u$ would yield a very similar value to $0.95(0.6\ F_y) = 0.57\ F_y$, and in fact the latter is even more conservative for HSS with specified nominal F_y/F_u ratios less than 0.79. Equation K2-4 need not be checked when $\beta > (1 - 1/\gamma)$ because this is the physical limit at which the branch can punch into (or out of) the main tubular member.

With round HSS in axially loaded K-connections, the size of the compression branch dominates the determination of the connection strength. Hence, the term D_b in Equation K2-6 pertains only to the compression branch and is not an average of the two branches. Thus, if one requires the connection strength expressed as a force in the tension branch, one can resolve the answer from Equation K2-6 into the direction of the tension branch, using Equation K2-8. That is, it is not necessary to repeat a calculation similar to Equation K2-6 with D_b as the tension branch. Note that Section K2.2c deals with branches subject to axial loading only. This is because there should only be axial forces in the branches of a typical planar K-connection if the truss structural analysis is performed according to one of the recommended methods, which are:

(i) pin-jointed analysis; or
(ii) analysis using web members pin-connected to continuous chord members, as shown in Figure C-K2.4.

3. Criteria for Rectangular HSS

The limits of validity in Section K2.3a generally represent the parameter range over which the design provisions have been verified in experiments. They are also

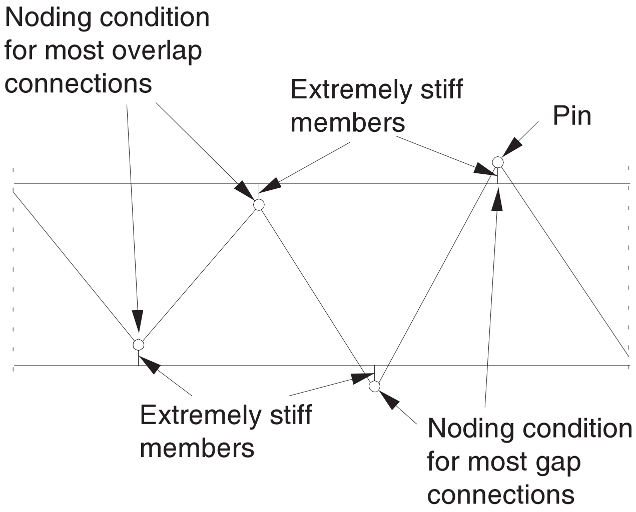


Fig. C-K2.4. Modeling assumption using web members pin-connected to continuous chord members.

set to eliminate the occurrence of certain failure modes for particular connection types, thereby making connection design easier. The following limitations from Section K2.3a bear explanation:

(2) The minimum branch angle is another practical limit for fabrication. Smaller branch angles are possible, but prior agreement with the fabricator should be made.

(8) The restriction on the minimum overlap is applied to ensure that there is an adequate interconnection of the branches to provide effective shear transfer from one branch to the other.

The restriction on the minimum gap ratio in Section K2.3c is modified from IIW (1989), according to Packer and Henderson (1997), to be more practical. The minimum gap size, g, is only specified so that adequate space is available to enable welding at the toes of the branches to be satisfactorily performed.

Equation K2-13 represents an analytical yield line solution for flexure of the connecting chord face. This nominal strength equation serves to limit connection deformations and is known to be well below the ultimate connection strength. A ϕ factor of 1.0 or Ω factor of 1.5 is thus appropriate. When the branch width exceeds 0.85 of the chord width this yield line failure mechanism will result in a noncritical design load.

The limit state of punching shear, evident in Equations K2-14 and K2-21, is based on the effective punching shear perimeter around the branch, with the total branch perimeter being an upper limit on this length. The term β_{eop} represents the chord face effective punching shear width ratio, adjacent to one (Equation K2-21) or two (Equation K2-14) branch walls transverse to the chord axis. This β_{eop} term incorporates a ϕ factor of 0.8 or Ω factor of 1.88. Applying to generally one dimension of the rectangular branch footprint, this was deemed by AWS to be similar to a global ϕ factor of 0.95 or Ω factor of 1.58 for the whole expression, so this expression for punching shear was implemented into AWS (2004) with an overall ϕ of 0.95. This ϕ factor of 0.95 or Ω factor of 1.58 has been carried over to this Specification and this topic is discussed further in Section K2.2. Notes below Equations K2-14 and K2-21 indicate when this failure mode is either physically impossible or noncritical. In particular, note that Equation K2-21 is noncritical for square HSS branches.

Equation K2-15 is generally in accord with a limit state given in IIW (1989), but with the k term [simply t in IIW (1989)] modified to be compatible with Equation K1-4, which in turn is derived from loads on I-shaped members. Equations K2-16 and K2-17 are in a format different than used internationally [for example, IIW (1989)] for this limit state and are unique to this Specification, having been replicated from Equations K1-5 and K1-6, along with their associated ϕ and Ω factors. These latter equations in turn are HSS versions (for two webs) of equations for I-shaped members with a single web.

The limit state of "uneven load distribution", which is manifested by local buckling of a compression branch or premature yield failure of a tension branch, represented by Equations K2-18 and K2-22, is checked by summing the effective areas of the four sides of the branch member. For T-, Y- and cross-connections the two walls of the branch transverse to the chord are likely to be only partially effective (Equation K2-18), whereas for gapped K-connections one wall of the branch transverse to the chord is likely to be only partially effective (Equation K-22). This reduced effectiveness is primarily a result of the flexibility of the connecting face of the chord, as incorporated in Equations K2-19 and K2-23. The effective width term b_{eoi} has been derived from research on transverse plate-to-HSS connections (as cited below for overlapped K-connections) and incorporates a ϕ factor of 0.8 or Ω factor of 1.88. Applying the same logic described above for the limit state of punching shear, a global ϕ factor of 0.95 or Ω factor of 1.58 was adopted in AWS D1.1 (AWS, 2004), and this has been carried over to this Specification [although, as noted previously, a ϕ factor of 1.0 is used in IIW (1989)].

For T-, Y- and cross-connections with $\beta \leq 0.85$, the connection strength is determined by Equation K2-13 only.

For axially loaded, gapped K-connections, plastification of the chord connecting face under the "push-pull" action of the branches is by far the most prevalent and critical failure mode. Indeed, if all the HSS members are square, this failure mode is critical and Equation K2-20 is the only one to be checked. This formula for chord face plastification is a semi-empirical "characteristic strength" expression, which has a confidence of 95 percent, taking into account the variation in experimental test results as well as typical variations in mechanical and geometric properties. Equation K2-20 is then multiplied by a ϕ factor for LRFD or divided by an Ω factor for ASD to further allow for the failure mode and provide an appropriate safety margin. A reliability calibration (Packer, Birkemoe, and Tucker, 1984) for this equation, using a database of 263 gapped K-connections and the exponential expression for the resistance factor (with a safety index of 3.0 and a coefficient of separation of 0.55) derived a ϕ factor of 0.89 (Ω factor of 1.69), while also imposing the parameter limits of validity. Since this failure mode dominates the test database, there is insufficient supporting test data to calibrate Equations K2-21 and K2-22.

For the limit state of shear yielding of the chord in the gap of gapped K-connections, Section K2.3c(c) differs from international practice [for example, IIW (1989)] but recommends application of another section of this Specification, Section G5. This limit state need only be checked if the chord member is rectangular (in other words, not square) and is also oriented such that the shorter wall of the chord section lies in the plane of the truss, hence providing a more critical chord shear condition due to the short "webs." The axial force present in the gap region of the chord member may also have an influence on the shear capacity of the chord webs in the gap region.

For K-connections, the scope covers both gapped and overlapped connections, although the latter are generally more difficult and more expensive to fabricate than

K-connections with a gap. However, an overlapped connection will, in general, produce a connection with a higher static strength, a stiffer truss, and a connection with a higher fatigue resistance, than its gapped connection counterpart. Note that Sections K2.3c and K2.3d deal with branches subject to axial loading only. This is because there should only be axial forces in the branches of a typical planar K-connection if the truss structural analysis is performed according to one of the recommended methods, which are:

(i) pin-jointed analysis, or
(ii) analysis using web members pin-connected to continuous chord members, as shown in Figure C-K2.4.

For rectangular HSS, the sole failure mode to be considered for design of overlapped connections is the limit state of "uneven load distribution" in the branches, manifested by either local buckling of the compression branch or premature yield failure of the tension branch. The design procedure presumes that one branch is welded solely to the chord and hence only has a single cut at its end. This can be considered "good practice" and the "thru member" is termed the overlapped member. For partial overlaps of less than 100 percent, the other branch is then double-cut at its end and welded to both the thru branch as well as the chord. The branch to be selected as the "thru" or overlapped member should be the one with the larger overall width. If both branches have the same width, the thicker branch should be the overlapped branch. For a single failure mode to be controlling (and not have failure by one branch punching into or pulling out of the other branch, for example), limits are placed on various connection parameters, including the relative width and relative thickness of the two branches. The foregoing fabrication advice for rectangular HSS also pertains to round HSS overlapped K-connections, but the latter involves more complicated profiling of the branch ends to provide good saddle fits.

Overlapped rectangular HSS K-connection strength calculations (Equations K2-24, K2-25 and K2-26) are performed initially just for the overlapping branch, regardless of whether it is in tension or compression, and then the resistance of the overlapped branch is determined from that. The equations for connection strength, expressed as a force in a branch, are based on the load-carrying contributions of the four side walls of the overlapping branch and follow the design recommendations of the International Institute of Welding (IIW, 1989; Packer and Henderson, 1997; AWS, 2004). The effective widths of overlapping branch member walls transverse to the chord (b_{eoi} and b_{eov}) depend on the flexibility of the surface on which they land, and are derived from plate-to-HSS effective width measurements (Rolloos, 1969; Wardenier, Davies, and Stolle, 1981; Davies and Packer, 1982). The constant of 10 in the b_{eoi} and b_{eov} terms has already been reduced from values determined in tests and incorporates a ϕ factor of 0.80 or Ω factor of 1.88 in those terms. Applying the same logic described above for the limit state of punching shear in T-, Y- and cross-connections, a global ϕ factor of 0.95 or Ω factor of 1.58 was adopted by AWS D1.1 and this has been carried over to this Specification [although as noted previously a ϕ factor of 1.0 is used by IIW (1989)].

The applicability of Equations K2-24, K2-25 and K2-26 depends on the amount of overlap, O_v, where $O_v = (q/p) \times 100\%$. It is important to note that p is the projected length (or imaginary footprint) of the overlapping branch on the connecting face of the chord, even though it does not physically contact the chord. Also, q is the overlap length measured along the connecting face of the chord beneath the region of overlap of the branches. This is illustrated in Figure C-K1.1.

A maximum overlap of 100 percent occurs when one branch sits completely on the other branch. In such cases, the overlapping branch is sometimes moved slightly up the overlapped branch so that the heel of the overlapping branch can be fillet welded to the face of the overlapped branch. If the connection is fabricated in this manner, an overlap slightly greater than 100 percent is created. In such cases, the connection strength for a rectangular HSS connection can be calculated by Equation K2-26 but with the B_{bi} term replaced by another b_{eov} term. Also, with regard to welding details, it has been found experimentally that it is permissible to just tack weld the "hidden toe" of the overlapped branch, providing that the components of the two branch member forces normal to the chord substantially balance each other. The "hidden toe" should be fully welded to the chord if the normal components of the two branch forces differ by more than 20 percent. If the components of the two branch forces normal to the chord do in fact differ significantly, the connection should also be checked for behavior as a T-, Y- or cross-connection, using the combined footprint and the net force normal to the chord (see Figure C-K2.1).

The design of "Welds to Branches" may be performed in either of two ways:

(a) The welds may be proportioned to develop the capacity of the connected branch wall, at all points along the weld length. This may be appropriate if the branch loading is complex or the loading is not known by the weld designer. Welds sized in this manner represent an upper limit on the required weld size and may be excessively conservative in some situations.

(b) The welds may be designed as "fit for purpose," to resist branch forces that are typically known in HSS truss-type connections. Many HSS truss web members have low axial loads, for a variety of possible reasons, and in such situations this weld design philosophy is ideal. However, the nonuniform loading of the weld perimeter due to the flexibility of the connecting HSS face must be taken into account by using weld effective lengths. Suitable effective lengths for various rectangular HSS connections subject to branch axial loading are given in Section K2.3e. These provisions are similar to those given in AWS (2004) and are based on full-scale HSS connection and truss tests that studied weld failures (Frater and Packer, 1992; 1992a; Packer and Cassidy, 1995). Adequate reliability is still obtained with the effective length expressions given if the directional strength increase allowed with fillet welds is used (Packer, 1995). Examples of weld joints in which weld effective lengths are less than 100 percent of the total weld length are shown in Figure C-K2.5. Most HSS trusses have the web members inclined to the chord at angles less than

50 degrees, in which cases the weld length around each branch perimeter in a K-connection will be 100 percent effective, as can be seen from Equation K2-31. Similar advice to that given in Section K2.3e is replicated in Section K1.3b for welds to transverse plates joined to rectangular HSS.

K3. HSS-TO-HSS MOMENT CONNECTIONS

Section K3 on HSS-to-HSS connections under moment loading is applicable to frames with PR or FR moment connections, such as Vierendeel girders. The provisions of Section K3 are not generally applicable to typical planar triangulated trusses (which are covered by Section K2), since the latter should be analyzed in a manner which results in no bending moments in the web members (see Commentary on Section K2). Thus, K-connections with moment loading on the branches are not covered by this Specification.

Available testing for HSS-to-HSS moment connections is much less extensive than that for axially-loaded T-, Y-, cross- and K-connections. Hence, the governing limit states to be checked for axially-loaded connections have been used as a basis for the possible limit states in moment-loaded connections. Thus, the design criteria for round HSS moment connections are based on the limit states of chord plastification and punching shear failure, with ϕ and Ω factors consistent with Section K2, while the design criteria for rectangular HSS moment connections are based on the limit states of plastification of the chord connecting face, chord side wall crushing, uneven load distribution and chord distortional failure, with ϕ and Ω factors consistent with Section K2. The "chord distortional failure" mode is applicable only to rectangular HSS T-connections with an out-of-plane bending moment on the branch. Rhomboidal distortion of the branch can be prevented by the use of

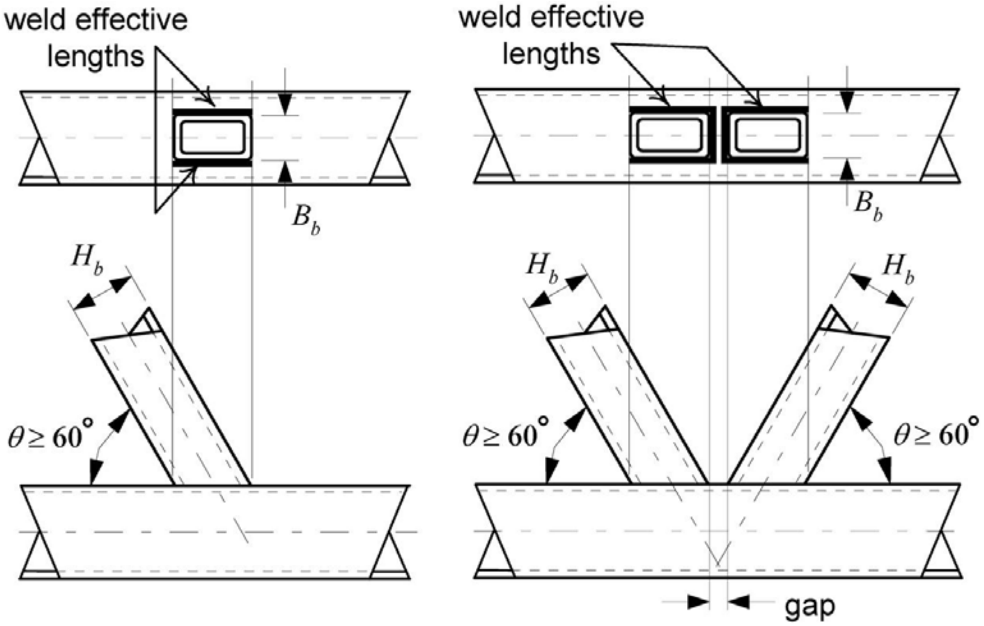


Fig. C-K2.5. Weld effective lengths for particular rectangular HSS connections.

stiffeners or diaphragms to maintain the rectangular cross-sectional shape of the chord. The limits of applicability of the equations in Section K3 are predominantly reproduced from Section K2. The basis for the equations in Section K3 is Eurocode 3 (2002), which represents one of the most up-to-date consensus specifications or recommendations on welded HSS-to-HSS connections. The equations in Section K3 have also been adopted in CIDECT Design Guide No. 9 (Kurobane and others, 2004).

CHAPTER L

DESIGN FOR SERVICEABILITY

L1. GENERAL PROVISIONS

Serviceability limit states are conditions in which the functions of a building are impaired because of local damage, deterioration or deformation of building components, or occupant discomfort. While serviceability limit states generally do not involve collapse of a building, loss of life or injury, they can seriously impair the building's usefulness and lead to costly repairs and other economic consequences. Serviceability provisions are essential to provide satisfactory performance of building structural systems. Neglect of serviceability may result in structures that are excessively flexible or otherwise perform unacceptably in service.

The three general types of structural behavior that are indicative of impaired serviceability in steel structures are:

(1) Excessive deflections or rotations that may affect the appearance, function or drainage of the building or may cause damaging transfer of load to nonstructural components and attachments;
(2) Excessive vibrations produced by the activities of the building occupants, mechanical equipment, or wind effects, which may cause occupant discomfort or malfunction of building service equipment; and
(3) Excessive local damage (local yielding, buckling, slip or cracking) or deterioration (weathering, corrosion and discoloration) during the service life of the structure.

Serviceability limit states depend on the occupancy or function of the building, the perceptions of its occupants, and the type of structural system. Limiting values of structural behavior intended to provide adequate levels of serviceability should be determined by a team consisting of the building owner/developer, the architect and the structural engineer after a careful analysis of all functional and economic requirements and constraints. In arriving at serviceability limits, the team should recognize that building occupants are able to perceive structural deformations, motions, cracking or other signs of distress at levels that are much lower than those that would indicate impending structural damage or failure. Such signs of distress may be viewed as an indication that the building is unsafe and diminish its economic value, and therefore must be considered at the time of design.

Service loads that may require consideration in checking serviceability include: (1) static loads from the occupants, snow or rain on the roof, or temperature fluctuations; and (2) dynamic loads from human activities, wind effects, the operation of mechanical or building service equipment, or traffic near the building. Service loads are loads that act on the structure at an arbitrary point in time, and may be only a fraction of the corresponding nominal load. The response of the structure to service loads generally can be analyzed assuming elastic behavior. Members that accumulate residual deformations under service loads also may require examination with respect to this long-term behavior.

Serviceability limit states and appropriate load combinations for checking conformance to serviceability requirements can be found in ASCE 7, *Minimum Design Loads for Buildings and Other Structures*, Appendix B, and the commentary to Appendix B (ASCE, 2002).

L2. CAMBER

Camber is frequently specified in order to provide a level surface under *permanent loads*, for reasons of appearance or for alignment with other work. In normal circumstances camber does nothing to prevent excessive deflection or vibration. Camber in trusses is normally created by adjustment of member lengths prior to making shop connections. It is normally introduced in beams by controlled heating of selected portions of the beam or by cold bending, or both. Designers should be aware of practical limits presented by normal fabricating and erection practices. The *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) provides tolerances on actual camber and recommends that all cambers be measured in the fabricating shop on unstressed members, along general guidelines. Further information on camber may be found in Ricker (1989).

L3. DEFLECTIONS

Excessive vertical deflections and misalignment arise primarily from three sources: (1) gravity loads, such as dead, live and snow loads; (2) effects of temperature, creep and differential settlement; and (3) construction tolerances and errors. Such deformations may be visually objectionable; cause separation, cracking or leakage of exterior cladding, doors, windows and seals; and cause damage to interior components and finishes. Appropriate limiting values of deformations depend on the type of structure, detailing and intended use (Galambos and Ellingwood, 1986). Historically, common deflection limits for horizontal members have been 1/360 of the span for floors subjected to reduced live load and 1/240 of the span for roof members. Deflections of about 1/300 of the span (for cantilevers, 1/150 of the length) are visible and may lead to general architectural damage or cladding leakage. Deflections greater than 1/200 of the span may impair operation of moveable components such as doors, windows and sliding partitions.

Deflection limits depend very much on the function of the structure and the nature of the supported construction. Traditional limits expressed as a fraction of the span length should not be extrapolated beyond experience. For example, the traditional limit of 1/360 of the span worked well for controlling cracks in plaster ceilings with spans common in the first half of the twentieth century. Many structures with more flexibility have performed satisfactorily with the now common, and more forgiving, ceiling systems. On the other hand, with the advent of longer structural spans, serviceability problems have been observed with flexible grid ceilings where actual deflections were far less than 1/360 of the span, because the distance between partitions or other elements that may interfere with ceiling deflection are far less than the span of the structural member. Proper control of deflections is a complex subject requiring careful application of professional judgment. West, Fisher, and Griffis (2003) provide an extensive discussion of the issues.

Deflection computations for composite beams should include an allowance for slip, creep and shrinkage (see Commentary Section I3.1).

In certain long-span floor systems, it may be necessary to place a limit (independent of span) on the maximum deflection to minimize the possibility of damage of adjacent nonstructural elements (ISO, 1977). For example, damage to nonload-bearing partitions may occur if vertical deflections exceed more than about $^3/_8$ in. (10 mm) unless special provision is made for differential movement (Cooney and King, 1988); however, many components can and do accept larger deformations.

Load combinations for checking static deflections can be developed using first-order reliability analysis (Galambos and Ellingwood, 1986). Current static deflection guidelines for floor and roof systems are adequate for limiting superficial damage in most buildings. A combined load with an annual probability of being exceeded of 5 percent is appropriate in most instances. For serviceability limit states involving visually objectionable deformations, repairable cracking or other damage to interior finishes, and other short-term effects, the suggested load combinations are:

$$D + L$$

$$D + 0.5S$$

For serviceability limit states involving creep, settlement or similar long-term or permanent effects, the suggested load combination is

$$D + 0.5L$$

The dead load effect, D, may be that portion of dead load that occurs following attachment of nonstructural elements. For example, in composite construction, the dead load effects frequently are taken as those imposed after the concrete has cured. For ceiling related calculations, the dead load effects may include only those loads placed after the ceiling structure is in place.

L4. DRIFT

Drift (lateral deflection) in a steel building is a serviceability issue primarily from the effects of wind. Drift limits are imposed on buildings to minimize damage to cladding and to nonstructural walls and partitions. Lateral frame deflection is evaluated for the building as a whole, where the applicable parameter is the *total building drift* (defined as the lateral frame deflection at the top of the most occupied floor divided by the height of the building to that level, Δ/H). For each floor, the applicable parameter is *interstory drift* [defined as the lateral deflection of a floor relative to the lateral deflection of the floor immediately below, divided by the distance between floors, $(\delta_n - \delta_{n-1})/h$].

Typical drift limits in common usage vary from $H/100$ to $H/600$ for *total building drift* and $h/200$ to $h/600$ for *interstory drift*, depending on building type and the type of cladding or partition materials used. The most widely used values are H (or h)/400 to H(or h)/500 (ASCE Task Committee on Drift Control of Steel Building Structures, 1988). An absolute limit on *interstory drift* is sometimes imposed by designers in light of evidence that damage to nonstructural partitions, cladding and glazing may occur if the *interstory drift* exceeds about 3/8 in. (10 mm), unless special detailing practices are employed to accommodate larger movements (Cooney and King, 1988; Freeman, 1977). Many components can accept deformations that are significantly larger. More specific information on the damage threshold for building materials is available in the literature (Griffis, 1993).

It is important to recognize that frame racking or shear distortion (in other words, strain) is the real cause of damage to building elements such as cladding and partitions. Lateral drift only captures the horizontal component of the racking and does not include potential vertical racking (as from differential column shortening in tall buildings), which also contributes to damage. Moreover, some lateral drift may be caused by rigid body rotation of the cladding or partition which by itself does not cause strain and therefore damage. A more precise parameter, the *drift damage index*, used to measure the potential damage, has been proposed (Griffis, 1993).

It must be emphasized that a reasonably accurate estimate of building drift is essential to controlling damage. The structural analysis must capture all significant components of potential frame deflection including flexural deformation of beams and columns, axial deformation of columns and braces, shear deformation of beams and columns, beam-column joint rotation (panel-zone deformation), the effect of member joint size, and the P-Δ effect (Charney, 1990). For many low rise steel frames with normal bay widths of 30 to 40 ft (9 to 12 m), use of center-to-center dimensions between columns without consideration of actual beam column joint size and panel zone effects will usually suffice for checking drift limits. The stiffening effect of nonstructural cladding, walls and partitions may be taken into account if substantiating information (stress versus strain behavior) regarding their effect is available.

The level of wind load used in drift limit checks varies among designers depending upon the frequency with which the potential damage can be tolerated. Some designers use the same nominal wind load (wind load specified by the building code without a load factor) as used for the strength design of the members (typically a 50 or 100 year mean recurrence interval wind load). Other designers use a 10 year or 20 year mean recurrence interval wind load (Griffis, 1993; ASCE, 2002). Use of factored wind loads (nominal wind load multiplied by the wind load factor) is generally considered to be very conservative when checking serviceability.

It is important to recognize that drift control limits by themselves in wind-sensitive buildings do not provide comfort of the occupants under wind load. See Section L6 for additional information regarding perception to motion in wind sensitive buildings.

L5. VIBRATION

The increasing use of high-strength materials with efficient structural systems and open plan architectural layouts leads to longer spans and more flexible floor systems having less damping. Therefore, floor vibrations have become an important design consideration. Acceleration is the recommended standard for evaluation.

An extensive treatment of vibration in steel-framed floor systems and pedestrian bridges is found in Murray and others (1997). This guide provides basic principles and simple analytical tools to evaluate steel-framed floor systems and footbridges for vibration serviceability due to human activities, including walking and rhythmic activities. Both human comfort and the need to control movement for sensitive equipment are considered.

L6. WIND-INDUCED MOTION

Designers of wind-sensitive buildings have long recognized the need for controlling annoying vibrations under the action of wind to protect the psychological well-being of the occupants (Chen and Robertson, 1972). The perception of building motion under the action of wind may be described by various physical quantities including maximum displacement, velocity, acceleration, and rate of change of acceleration (sometimes called “jerk”). Acceleration has become the standard for evaluation because it is readily measured in the field and can be easily calculated analytically. Human response to building motion is a complex phenomenon involving many psychological and physiological factors. Perception and tolerance thresholds of acceleration as a measure of building motion are known to depend on factors such as frequency of the building, occupant gender, age, body posture (sitting, standing or reclining), body orientation, expectation of motion, body movement, visual cues, acoustic clues, and the type of motion (translational or torsional) (ASCE, 1981). Different thresholds and tolerance levels exist for different people and responses can be very subjective. It is known that some people can become accustomed to building motion and tolerate higher levels than others. Limited research exists on this subject but certain standards have been applied for design as discussed below.

Acceleration in wind-sensitive buildings may be expressed as either root mean square (RMS) or peak acceleration. Both measures are used in practice and there is no clear agreement as to which is the more appropriate measure of motion perception. Some researchers believe that peak acceleration during wind storms is a better measure of actual perception but that RMS acceleration during the entire course of a wind storm is a better measure of actual discomfort. Target peak accelerations of 21 milli-g (0.021 times the acceleration of gravity) for commercial buildings (occupied mostly during daylight hours) and 15 milli-g for residential buildings (occupied during the entire day) under a 10-year mean recurrence interval wind storm have been successfully used in practice for many tall building designs (Griffis, 1993). The target is generally more strict for residential buildings because of the continuous occupancy, the perception that people are less sensitive and more tolerant at work than at home, the fact that there is more turnover in commercial buildings, and the fact that commercial buildings are more easily evacuated for peak wind events. Peak acceleration and RMS acceleration in wind sensitive buildings are related by the "peak factor" best determined in a wind tunnel study and generally in the range of 3.5 for tall buildings (in other words, peak acceleration = peak factor × RMS acceleration). Guidance for design acceleration levels used in building design may be found in the literature (Chen and Robertson, 1972; Griffis, 1993; Hansen and Reed, 1973; Irwin, 1986; NRCC, 1990).

It is important to recognize that perception to building motion is strongly influenced by building mass and available damping as well as stiffness (Vickery, Isyumov, and Davenport, 1983). For this reason, building drift limits by themselves should not be used as the sole measure of controlling building motion (Islam, Ellingwood, and Corotis, 1990). Damping levels for use in evaluating building motion under wind events are generally taken as approximately 1 percent of critical damping for steel buildings.

L7. EXPANSION AND CONTRACTION

The satisfactory accommodation of expansion and contraction cannot be reduced to a few simple rules, but must depend largely upon the judgment of a qualified engineer.

The problem is likely to be more serious in buildings with masonry walls than with prefabricated units. Complete separation of the framing at widely spaced expansion joints is generally more satisfactory than more frequently located devices that depend upon the sliding of parts in bearing, and usually less expensive than rocker or roller expansion bearings.

Creep and shrinkage of concrete and yielding of steel are among the causes, other than temperature, for dimensional changes. Conditions during construction, such as temperature effects before enclosure of the structure, should also be considered.

Guidelines for the recommended size and spacing of expansion joints in buildings may be found in NRC (1974).

L8. CONNECTION SLIP

In bolted connections with bolts in holes having only small clearances, such as standard holes and slotted holes loaded transversely to the axis of the slot, the amount of possible slip is small. Slip at these connections is not likely to have serviceability implications. Possible exceptions include certain unusual situations where the effect of slip is magnified by the configuration of the structure, such as a connection at the base of a shallow cantilever beam or post where a small amount of bolt slip may produce unacceptable rotation and deflection.

This Specification requires that connections with oversized holes or slotted holes loaded parallel to the axis of the slot be designed as slip-critical connections. For a discussion of slip at these connections see the Commentary to Section J3.8. Where slip at service loads is a realistic possibility in these connections, the effect of connection slip on the serviceability of the structure must be considered.

CHAPTER M

FABRICATION, ERECTION AND QUALITY CONTROL

M1. SHOP AND ERECTION DRAWINGS

Supplementary information relevant to shop drawing documentation and associated fabrication, erection and inspection practices may be found in the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005) and in Schuster (1997).

M2. FABRICATION

1. Cambering, Curving and Straightening

The use of heat for straightening or cambering members is permitted for A514/A514M and A852/A852M steel, as it is for other steels. However, the maximum temperature permitted is 1,100 °F (590 °C) compared to 1,200 °F (650 °C) for other steels.

Cambering of flexural members, when required by the contract documents, may be accomplished in various ways. In the case of trusses and girders, the desired curvature can be built in during assembly of the component parts. Within limits, rolled beams can be cold-cambered.

Local application of heat has long been used as a means of straightening or cambering beams and girders. The method depends upon an ultimate shortening of the heat-affected zones. A number of such zones, on the side of the member that would be subject to compression during cold-cambering or "gagging," are heated enough to be "upset" by the restraint provided by surrounding unheated areas. Shortening takes place upon cooling.

While the final curvature or camber can be controlled by these methods, it must be realized that some deviation due to workmanship considerations and permanent change due to handling is inevitable. Camber is usually defined by one mid-ordinate, as control of more than one point is difficult and not normally required. Reverse cambers are difficult to achieve and are discouraged. Long cantilevers are sensitive to camber and may deserve closer control.

2. Thermal Cutting

Thermal cutting is preferably done by machine. The requirement for a positive preheat of 150 °F (66 °C) minimum when beam copes and weld access holes are thermally cut in ASTM A6/A6M hot-rolled shapes with a flange thickness exceeding 2 in. (50 mm) and in built-up shapes made of material more than 2 in.

(50 mm) thick tends to minimize the hard surface layer and the initiation of cracks. This requirement for preheat for thermal cutting does not apply when the radius portion of the access hole or cope is drilled and the thermally cut portion is essentially linear. Such thermally cut surfaces are required to be ground and inspected in accordance with Section J1.6.

4. Welded Construction

To avoid weld contamination, the light oil coating that is generally present after manufacturing an HSS should be removed with a suitable solvent in locations where welding will be performed. In cases where an external coating has been applied at the mill, the coating should be removed at the location of welding or the manufacturer should be consulted regarding the suitability of welding in the presence of the coating.

5. Bolted Construction

In most connections made with high-strength bolts, it is only required to install the bolts to the snug-tight condition. This includes bearing-type connections where slip is permitted and, for ASTM A325 or A325M bolts only, tension (or combined shear and tension) applications where loosening or fatigue due to vibration or load fluctuations are not design considerations.

It is suggested that snug-tight bearing-type connections with ASTM A325 or A490 bolts be used in applications where A307 bolts are permitted.

This section provides rules for the use of oversized and slotted holes paralleling the provisions that have been in the RCSC Specification since 1972 (RCSC, 2004), extended to include A307 bolts, which are outside the scope of the RCSC Specification.

The Specification previously limited the methods used to form holes, based on common practice and equipment capabilities. Fabrication methods have changed and will continue to do so. To reflect these changes, this Specification has been revised to define acceptable quality instead of specifying the method used to form the holes, and specifically to permit thermally cut holes. AWS C4.7, Sample 3, is useful as an indication of the thermally cut profile that is acceptable (AWS, 1977). The use of numerically controlled or mechanically guided equipment is anticipated for the forming of thermally cut holes. To the extent that the previous limits may have related to safe operation in the fabrication shop, fabricators are referred to equipment manufacturers for equipment and tool operating limits.

10. Drain Holes

Because the interior of an HSS is difficult to inspect, concern is sometimes expressed regarding internal corrosion. However, good design practice can eliminate the concern and the need for expensive protection.

Corrosion occurs in the presence of oxygen and water. In an enclosed building, it is improbable that there would be sufficient reintroduction of moisture to cause severe corrosion. Therefore, internal corrosion protection is a consideration only in HSS that are exposed to weather.

In a sealed HSS, internal corrosion cannot progress beyond the point where the oxygen or moisture necessary for chemical oxidation is consumed (AISI, 1970). The oxidation depth is insignificant when the corrosion process must stop, even when a corrosive atmosphere exists at the time of sealing. If fine openings exist at connections, moisture and air can enter the HSS through capillary action or by aspiration due to the partial vacuum that is created if the HSS is cooled rapidly (Blodgett, 1967). This can be prevented by providing pressure-equalizing holes in locations that make it impossible for water to flow into the HSS by gravity.

Situations where an internal protective coating may be required include: (1) open HSS where changes in the air volume by ventilation or direct flow of water is possible; and (2) open HSS subject to a temperature gradient that causes condensation. In such instances it may also be prudent to use a minimum $^5/_{16}$ in. (8 mm) wall thickness.

HSS that are filled or partially filled with concrete should not be sealed. In the event of fire, water in the concrete will vaporize and may create pressure sufficient to burst a sealed HSS. Care should be taken to ensure that water does not remain in the HSS during or after construction, since the expansion caused by freezing can create pressure that is sufficient to burst an HSS.

Galvanized HSS assemblies should not be completely sealed because rapid pressure changes during the galvanizing process tend to burst sealed assemblies.

11. Requirements for Galvanized Members

Cracking has been observed in steel members during hot-dip galvanizing. The occurrence of these cracks has been correlated to several characteristics including, but not limited to, highly restrained details, base material chemistry, galvanizing practices, and fabrication workmanship. The requirement to grind beam copes before galvanizing will not prevent all cope cracks from occurring during galvanizing. However, it has been shown to be an effective means to reduce the occurrence of this phenomenon.

Galvanizing of structural steel and hardware such as fasteners is a process that depends on special design detailing and fabrication to achieve the desired level of corrosion protection. ASTM publishes a number of standards relating to galvanized structural steel:

ASTM A123 (ASTM, 2002) provides a standard for the galvanized coating and its measurement and includes provisions for the materials and fabrication of the products to be galvanized.

ASTM A153 (ASTM, 2001) is a standard for galvanized hardware such as fasteners that are to be centrifuged.

ASTM A384 (ASTM, 2002a) is the Standard Practice for Safeguarding Against Warpage and Distortion During Hot-Dip Galvanizing. It includes information on factors that contribute to warpage and distortion as well as suggestions for correction for fabricated assemblies.

ASTM A385 (ASTM, 2001a) is the Standard Practice for Providing High Quality Zinc coatings. It includes information on base materials, venting, treatment of contacting surfaces, and cleaning. Many of these provisions should be indicated on design and detail drawings.

ASTM A780 (ASTM, 2001b) provides for repair of damaged and uncoated areas of hot-dip galvanized coatings.

M3. SHOP PAINTING

1. General Requirements

The surface condition of unpainted steel framing of long-standing buildings that have been demolished has been found to be unchanged from the time of its erection, except at isolated spots where leakage may have occurred. Even in the presence of leakage, the shop coat is of minor influence (Bigos, Smith, Ball, and Foehl, 1954).

This Specification does not define the type of paint to be used when a shop coat is required. Final exposure and individual preference with regard to finish paint are factors that determine the selection of a proper primer. A comprehensive treatment of the subject is found in SSPC (2000).

3. Contact Surfaces

Special concerns regarding contact surfaces of HSS should be considered. As a result of manufacturing, a light oil coating is generally present on the outer surface of the HSS. If paint is specified, HSS must be cleaned of this oil coating with a suitable solvent; see SSPC (2000).

5. Surfaces Adjacent to Field Welds

This Specification allows for welding through surface materials, including appropriate shop coatings that do not adversely affect weld quality nor create objectionable fumes.

M4. ERECTION

2. Bracing

For information on the design of temporary lateral support systems and components for low-rise buildings see Fisher and West (1997).

4. Fit of Column Compression Joints and Base Plates

Tests on spliced full-size columns with joints that had been intentionally milled out-of-square, relative to either strong or weak axis, demonstrated that the load-carrying capacity was the same as that for similar columns without splices (Popov and Stephen, 1977). In the tests, gaps of 1/16 in. (2 mm) were not shimmed; gaps of 1/4 in. (6 mm) were shimmed with nontapered mild steel shims. Minimum size partial-joint-penetration groove welds were used in all tests. No tests were performed on specimens with gaps greater than 1/4 in. (6 mm).

5. Field Welding

The purpose of wire brushing shop paint on surfaces adjacent to joints to be field welded is to reduce the possibility of porosity and cracking and also to reduce any environmental hazard. Although there are limited tests that indicate that painted surfaces result in sound welds without wire brushing, other tests have resulted in excessive porosity and/or cracking when welding coated surfaces. Wire brushing to reduce the paint film thickness minimizes weld rejection. Grinding or other treatment beyond wire brushing is not necessary.

M5. QUALITY CONTROL

To facilitate quality control, inspection, and identification, reference should be made to the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005).

5. Identification of Steel

Material identification procedures should be sufficient to show the material specification designations and to tie the material to any special material requirements, such as notch toughness when specified.

APPENDIX 1

INELASTIC ANALYSIS AND DESIGN

1.1. GENERAL PROVISIONS

The design of statically indeterminate steel structures according to Appendix 1 is based on their *inelastic strength*. Although design could be performed according to Section B3.4 (ASD) if the appropriate load factor were included in the analysis, this process is more complicated than simply performing design according to Section B3.3 (LRFD). For this reason, only LRFD provisions are provided. An exception is permitted in Section 1.3, as discussed below.

1.2. MATERIALS

Extensive past research on the plastic and inelastic behavior of continuous beams, *rigid frames* and connections has amply demonstrated the suitability of steel with yield stress levels up to 65 ksi (450 MPa) (ASCE, 1971).

1.3. MOMENT REDISTRIBUTION

The provision of Section 1.3 has been a part of the Specification since the 1949 edition. The permission of applying a redistribution of 10 percent of the elastically calculated bending moment at points of interior support due to gravity loading on continuous compact beams gives partial recognition to the philosophy of plastic design. Figure C-A-1.1 illustrates the application of this provision by comparing calculated moment diagrams with the diagrams altered by this provision.

1.4. LOCAL BUCKLING

Inelastic design requires that, up to the formation of the plastic mechanism or up to the peak of the inelastic load-deflection curve, the moments at the plastic hinge locations remain at the level of the plastic moment. This implies that the member must have sufficient inelastic rotation capacity to permit the redistribution of the moments. Sections that are designated as compact in Section B4 have a rotation capacity of approximately 3 and are suitable for plastic design. The limiting width/thickness ratio designated as λ_r in Table B4.1 is the maximum slenderness ratio for this rotation capacity to be achieved. Further discussion of the antecedents of these provisions is given in Commentary Section B4.

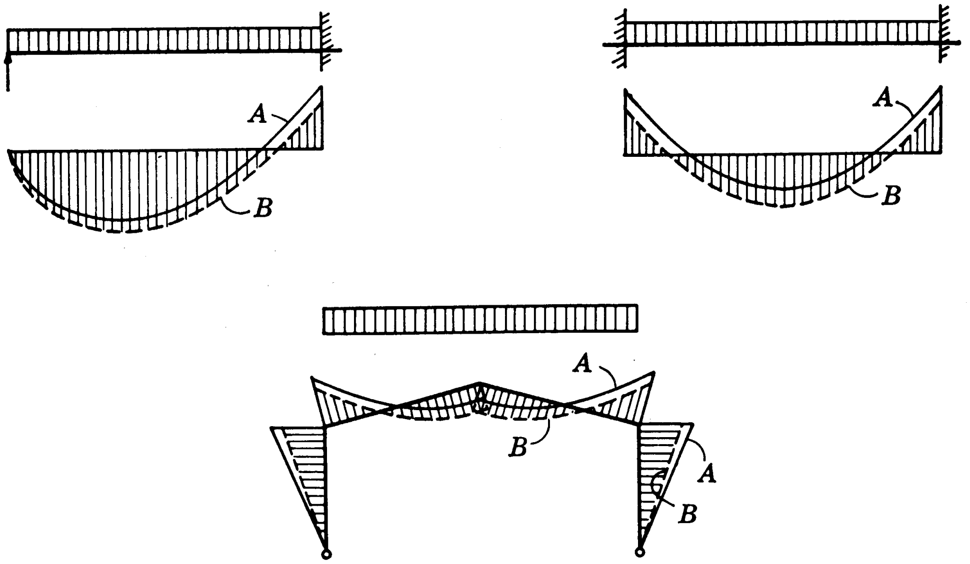
The additional slenderness limits in Equations A-1-1 through A-1-4 apply to cases not covered in Table B4.1. The equations for height-to-thickness ratio limits of webs of wide-flange members and rectangular HSS under combined flexure and compression have been taken from Table B5.1 of the 1999 *LRFD Specification* (AISC, 2000b). These provisions have been part of the plastic design requirements since the 1969 Specification, and they are based on research documented in *Plastic Design in Steel, A Guide and a Commentary* (ASCE, 1971). The equations for the flanges of HSS and other boxed sections (Equation A-1-3) and for round HSS (Equation A-1-4) are from the *Specification for the Design of Steel Hollow Structural Sections* (AISC, 2000).

The use of single-angle, tee and double-angle sections in statically indeterminate beams in plastic design is not recommended since the inelastic rotation capacity in the regions where the moment produces compression in an outstanding leg will typically not be sufficient.

1.5. STABILITY AND SECOND-ORDER EFFECTS

Section 1.5 requires that the equilibrium equations be formulated on the geometry of the deformed structure for frames designed by plastic or inelastic analysis.

Continuous, braced beams not subject to axial loads can be designed by *first-order plastic analysis*. Braced frames and moment frames having small axial loads in the members that are braced to prevent lateral-torsional buckling and loaded so as to produce bending about the major axis only may also be designed by first-order plastic analysis, provided that the requirements of Chapters C (the B_1 and B_2 amplification factors), E (column equations) and H (interaction equations) are accounted for. First-order plastic analysis is treated in ASCE (1971), in steel design textbooks [for example, Salmon and Johnson (1996) and Galambos, Lin, and Johnston (1996)], in textbooks dedicated entirely to plastic design [for example: Horne and Morris, (1982); Chen and Sohal (1995); and Bruneau, Uang, and



A = Actual moment diagram
B = Modified diagram corresponding to 10 percent moment reduction allowance at interior supports

Fig. C-A-1.1. Examples of effects of 10 percent moment redistribution.

Whittaker (1998)] and in structural engineering handbooks (Gaylord, Gaylord, and Stallmeyer, 1997).

First-order plastic analysis is applicable to continuous beams and low-rise frames with small axial loads. For these simple structures the tools of plastic analysis are readily available to the designer from books giving simple ways of calculating the plastic mechanism loads. This is not so for the case of general moment frames, where a full second-order inelastic analysis must be performed for the determination of the load effects on the members and the connections. The state-of-the-art of inelastic frame analysis is discussed in Chapter 16 of Galambos (1998). Textbooks [for example, Chen and Sohal (1995) and McGuire, Gallagher, and Ziemian (2000)] present the basic approaches to inelastic analysis, as well as worked examples and computer programs for use by students studying the subject.

1. Braced Frames

In Section 1.5.1 two constraints are given for the plastic design of braced frames: (1) the bracing system shall remain elastic; and (2) the axial force in any column must not exceed 85 percent of the *squash load*, $F_y A_g$.

2. Moment Frames

The provision in Section 1.5.2 restricts the axial force in any column to 75 percent of the *squash load*. This provision, as well as the corresponding one in Section 1.5.1, is a cautionary limitation because at high levels of axial force insufficient research has been conducted to ensure that sufficient inelastic rotation capacity remains in the member.

1.6. COLUMNS AND OTHER COMPRESSION MEMBERS

Columns in braced frames and moment frames that are designed on the basis of first-order inelastic analysis or a plastic mechanism analysis are proportioned according to the requirements of Section E3, with an effective length determined by methods of stability analysis. For moment frames, the effective length may exceed unity.

1.7. BEAMS AND OTHER FLEXURAL MEMBERS

The plastic moment, M_p, is the maximum moment that acts at the plastic hinge. When a wide-flange member is subject to flexure about its major axis, the ratio of the plastic moment to the yield moment is approximately 1.1 to 1.2. However, if flexure is about the minor axis, this ratio can exceed 1.6. A limit of $1.6M_y$ is imposed in order to prevent excessive yielding under service loads.

Portions of members that would be required to rotate inelastically as a plastic hinge, while the moments are redistributed to eventually form a plastic mechanism, need more closely spaced bracing than similar parts of a continuous frame designed in accordance with elastic theory. Equations A-1-7 and A-1-8 define the maximum permitted unbraced length in the vicinity of plastic hinges for wide-flange shapes bent about their major axis, and for rectangular shapes and symmetric box beams, respectively. These equations are identical to those in the 1999 *LRFD Specification* (AISC, 1999). They are different from the corresponding equations in Chapter N of the 1989 *ASD Specification* (AISC, 1989). The new equations are based on research reported in Yura and others (1978).

Some requirements that were in the plastic design chapter of the 1989 *ASD Specification* (AISC, 1989) are no longer explicitly enumerated in Appendix 1. One of these is the provision that web stiffeners are required at a point of load application where a plastic hinge would form. However, the provisions of Section J10 apply for plastic as well as elastic design. No mention is made of shear requirements, but the requirements of Chapter G apply. The plastic shear strength is $V_p = V_n = 0.6F_yA_w$ (Equation G2-1, with C_v equal to 1.0). The maximum permitted plastic web slenderness limit for plastic design is thus equal to

$$(h/t_w)_p = 1.1\sqrt{k_vE/F_y} = 1.1\sqrt{5E/F_y} = 2.5\sqrt{E/F_y} \qquad \text{(C-A-1-1)}$$

with a shear buckling coefficient $k_v = 5$. The plastic shear strength of $0.6F_yA_w$ is a liberalization of the previously used $0.55F_yA_w$ that was recommended in ASCE (1971) based on extensive research.

1.8. MEMBERS UNDER COMBINED FORCES

Members subject to bending moment and axial force are subject to the provisions of the interaction equations in Section H1. If the member contains a plastic hinge within its span or at its end, and bending is about the major axis of a doubly symmetric section, then the member must be laterally braced near the hinge location (Equation A-1-7 or A-1-8). When the unbraced length of the member exceeds these limits, the inelastic rotation capacity may be impaired, due to the combined influence of lateral and torsional deformation, to such an extent that plastic action is not achievable. However, if the required moment is small enough so the limitations of the interaction equations in Section H1 are fulfilled, the member will be strong enough to function at a joint where required hinge action is provided in another member entering the joint. If the forces on the beam-column include torsion, plastic design is not permitted by this Specification.

1.9. CONNECTIONS

The connections adjacent to plastic hinges must be designed with sufficient strength and ductility to sustain the forces and deformations imposed under the required loads. The practical implementation of this rule is that the applicable requirements of Chapter J must be strictly adhered to. The provisions for connection design in Chapter J have been developed from plasticity theory and verified by extensive testing, as discussed in ASCE (1971) and in many books and papers. Thus the connections that meet these provisions are inherently qualified for use in plastically designed structures.

APPENDIX 2

DESIGN FOR PONDING

Ponding stability is determined by ascertaining that the conditions of Equations A-2-1 and A-2-2 of Appendix 2 are fulfilled. These equations provide a conservative evaluation of the stiffness required to avoid runaway deflection, giving a factor of safety of four against ponding instability.

Since Equations A-2-1 and A-2-2 yield conservative results, it may be advantageous to perform a more detailed stress analysis to check whether a roof system that does not meet the above equations is still safe against ponding failure.

For the purposes of Appendix 2, *secondary members* are the beams or joists that directly support the distributed ponding loads on the roof of the structure, and *primary members* are the beams or girders that support the concentrated reactions from the *secondary members* framing into them. Representing the deflected shape of the primary and critical *secondary member* as a half-sine wave, the weight and distribution of the ponded water can be estimated, and, from this, the contribution that the deflection each of these members makes to the total ponding deflection can be expressed as follows (Marino, 1966):

For the *primary member*

$$\Delta_w = \frac{\alpha_p \Delta_o \left[1 + 0.25\pi\alpha_s + 0.25\pi\rho(1+\alpha_s)\right]}{1 - 0.25\pi\alpha_p\alpha_s} \qquad \text{(C-A-2-1)}$$

For the *secondary member*

$$\delta_w = \frac{\alpha_s \delta_o \left[1 + \dfrac{\pi^2}{32}\alpha_p + \dfrac{\pi^2}{8\rho}(1+\alpha_p) + 0.185\alpha_s\alpha_p\right]}{1 - 0.25\pi\alpha_p\alpha_s} \qquad \text{(C-A-2-2)}$$

In these expressions Δ_o and δ_o are, respectively, the primary and secondary beam deflections due to loading present at the initiation of ponding, and

$$\alpha_p = C_p / \left(1 - C_p\right)$$

$$\alpha_s = C_s / (1 - C_s)$$

$$\rho = \delta_o / \Delta_o = C_s / C_p$$

Using the above expressions for Δ_w and δ_w, the ratios Δ_w / Δ_o and δ_w / δ_o can be computed for any given combination of primary and secondary beam framing using the computed values of parameters C_p and C_s, respectively, defined in the Specification.

Even on the basis of unlimited elastic behavior, it is seen that the ponding deflections would become infinitely large unless

$$\left(\frac{C_p}{1-C_p}\right)\left(\frac{C_s}{1-C_s}\right) < \frac{4}{\pi} \qquad \text{(C-A-2-3)}$$

Since elastic behavior is not unlimited, the effective bending strength available in each member to resist the stress caused by ponding action is restricted to the difference between the yield stress of the member and the stress f_o produced by the total load supported by it before consideration of ponding is included.

Note that elastic deflection is directly proportional to stress. The admissible amount of ponding in either the primary or critical (midspan) *secondary member*, in terms of the applicable ratio Δ_w/Δ_o and δ_w/δ_o, can be represented as $(0.8F_y - f_o)/f_o$, assuming a factor of safety of 1.25 against yielding under the ponding load. Substituting this expression for Δ_w/Δ_o and δ_w/δ_o, and combining with the foregoing expressions for Δ_w and δ_w, the relationship between the critical values for C_p and C_s and the available elastic bending strength to resist ponding is obtained. The curves presented in Figures A-2.1 and A-2.2 are based upon this relationship. They constitute a design aid for use when a more exact determination of required flat roof framing stiffness is needed than given by the Specification provision that $C_p + 0.9C_s \le 0.25$.

Given any combination of primary and secondary framing, the stress index is computed as follows:

For the *primary member*

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_p \qquad \text{(C-A-2-4)}$$

For the *secondary member*

$$U_p = \left(\frac{0.8F_y - f_o}{f_o}\right)_s \qquad \text{(C-A-2-5)}$$

where

f_o = the stress due to $D + R$ (D = nominal dead load, R = nominal load due to rainwater or ice exclusive of the ponding contribution), ksi (MPa)

Depending upon geographic location, this loading should include such amount of snow as might also be present, although ponding failures have occurred more frequently during torrential summer rains when the rate of precipitation exceeded the rate of drainage runoff and the resulting hydraulic gradient over large roof areas caused substantial accumulation of water some distance from the eaves.

Given the size, spacing, and span of a tentatively selected combination of primary and secondary beams, for example, one may enter Figure A-2.1 at the level of the computed stress index U_p, determined for the primary beam; move horizontally to the computed C_s value of the secondary beams; then move downward to the abscissa scale. The combined stiffness of the primary and secondary framing is sufficient to prevent ponding if the flexibility constant read from this latter scale is larger than the value of C_p computed for the given *primary member*; if not, a stiffer primary or secondary beam, or combination of both, is required.

If the roof framing consists of a series of equally spaced wall-bearing beams, the beams would be considered as *secondary members*, supported on an infinitely stiff *primary member*. For this case, one would use Figure A-2.2. The limiting value of C_s would be determined by the intercept of a horizontal line representing the U_s value and the curve for $C_p = 0$.

The ponding deflection contributed by a metal deck is usually such a small part of the total ponding deflection of a roof panel that it is sufficient merely to limit its moment of inertia [in.^{4} per foot (mm^4 per meter) of width normal to its span] to 0.000025 (3940) times the fourth power of its span length, as provided in Equation A-2-2. However, the stability against ponding of a roof consisting of a metal roof deck of relatively slender depth-span ratio, spanning between beams supported directly on columns, may need to be checked. This can be done using Figures A-2.1 or A-2.2 with the following computed values:

U_p = stress index for the supporting beam
U_s = stress index for the roof deck
C_p = flexibility constant for the supporting beams
C_s = flexibility constant for one foot width of the roof deck ($S = 1.0$)

Since the shear rigidity of the web system is less than that of a solid plate, the moment of inertia of steel joists and trusses should be taken as somewhat less than that of their chords (Heinzerling, 1987).

APPENDIX 3

DESIGN FOR FATIGUE

When the limit state of fatigue is a design consideration, its severity is most significantly affected by the number of load applications, the magnitude of the stress range, and the severity of the stress concentrations associated with particular details. Issues of fatigue are not normally encountered in building design; however, when encountered and if the severity is great enough, fatigue is of concern and all provisions of Appendix 3 must be satisfied.

3.1. GENERAL

In general, members or connections subject to less than a few thousand cycles of loading will not constitute a fatigue condition except possibly for cases involving full reversal of loading and particularly sensitive categories of details. This is because the applicable cyclic design stress range will be limited by the static design stress. At low levels of cyclic tensile stress, a point is reached where the stress range is so low that fatigue cracking will not initiate regardless of the number of cycles of loading. This level of stress is defined as the *fatigue threshold*, F_{TH}.

Extensive test programs using full-size specimens, substantiated by theoretical stress analysis, have confirmed the following general conclusions (Fisher, Frank, Hirt, and McNamee, 1970; Fisher, Albrecht, Yen, Klingerman, and McNamee, 1974):

(1) Stress range and notch severity are the dominant stress variables for welded details and beams;
(2) Other variables such as minimum stress, mean stress, and maximum stress are not significant for design purposes; and
(3) Structural steels with yield points of 36 to 100 ksi (250 to 690 MPa) do not exhibit significantly different fatigue strengths for given welded details fabricated in the same manner.

3.2. CALCULATION OF MAXIMUM STRESSES AND STRESS RANGES

Fluctuation in stress that does not involve tensile stress does not cause crack propagation and is not considered to be a fatigue situation. On the other hand, in elements of members subject solely to calculated compressive stress, fatigue cracks may initiate in regions of high tensile *residual stress*. In such situations, the cracks generally do not propagate beyond the region of the residual tensile stress, because the *residual stress* is relieved by the crack. For this reason, stress ranges that are completely in compression need not be investigated for fatigue. For cases involving cyclic reversal of stress, the calculated stress range must be taken as the sum of the compressive stress and the tensile stress caused by different directions or patterns of the applied live load.

3.3. DESIGN STRESS RANGE

Fatigue resistance has been derived from an exponential relationship between the number of cycles to failure N and the stress range, S_r, called an $S - N$ relationship, of the form

$$N = \frac{C_f}{S_r^n} \qquad \text{(C-A-3-1)}$$

The general relationship is often plotted as a linear log-log function ($Log\ N = A - n\ Log\ S_r$). Figure C-A-3.1 shows the family of fatigue resistance curves identified as Categories A, B, B′, C, C′, D, E and E′. These relationships were established based on an extensive database developed in the United States and abroad (Keating and Fisher, 1986). The design stress range has been developed by adjusting the coefficient C_f so that a design curve is provided that lies two standard deviations of the standard error of estimate of the fatigue cycle life below the mean $S - N$ relationship of the actual test data. These values of C_f correspond to a probability of failure of 2.5 percent of the design life.

Prior to the 1999 AISC *Load and Resistance Factor Design Specification for Structural Steel Buildings* (AISC, 2000b), stepwise tables meeting the above criteria of cycles of loading, stress categories and design stress ranges were provided in the specifications. A single table format (Table A-3.1) was introduced in the 1999 AISC *LRFD Specification* that provides the stress categories, ingredients

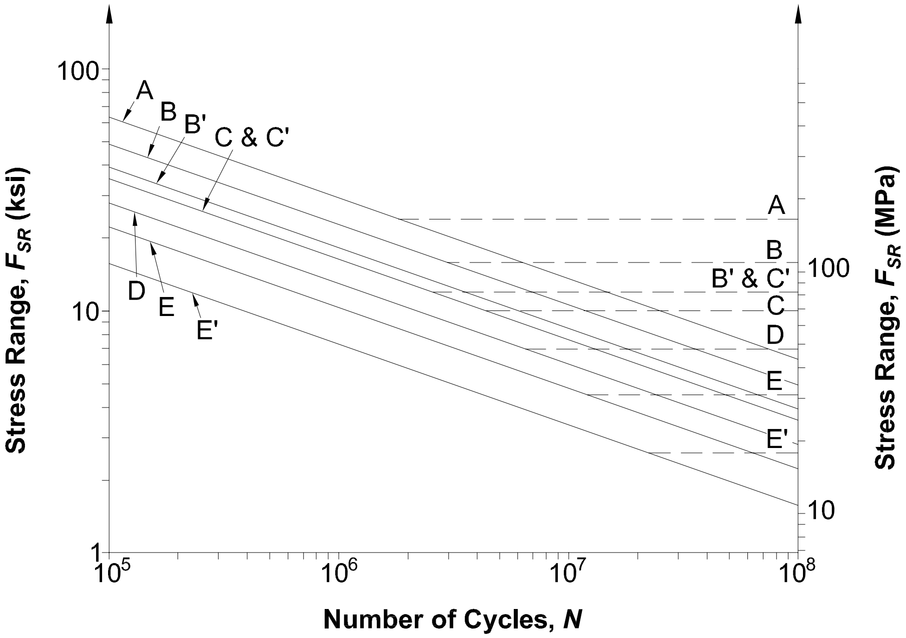


Fig. C-A-3.1. Fatigue resistance curves.

for the applicable equation, and information and examples including the sites of concern for potential crack initiation (AISC, 2000b).

Table A-3.1 is organized into 8 sections of general conditions for fatigue design, as follows:

- Section 1 provides information and examples for the steel material at copes, holes, cutouts or as produced.
- Section 2 provides information and examples for various types of mechanically fastened joints including eyebars and pin plates.
- Section 3 provides information related to welded connections used to join built-up members, such as longitudinal welds, access holes and reinforcements.
- Section 4 deals only with longitudinal load carrying fillet welds at shear splices.
- Section 5 provides information for various types of groove and fillet welded joints that are transverse to the applied cyclic stress.
- Section 6 provides information on a variety of groove welded attachments to flange tips and web plates as well as similar attachments connected with either fillet or partial-joint-penetration groove welds.
- Section 7 provides information on several short attachments to structural members.
- Section 8 collects several miscellaneous details such as shear connectors, shear on the throat of fillet, plug and slot welds, and their impact on base metal. It also provides for tension on the stress area of various bolts, threaded anchor rods and hangers.

A similar format and consistent criteria are used by other specifications.

When fabrication details involving more than one stress category occur at the same location in a member, the stress range at that location must be limited to that of the most restrictive category. The need for a member larger than required by static loading will often be eliminated by locating notch-producing fabrication details in regions subject to smaller ranges of stress.

A detail not explicitly covered before 1989 was added in 1999 to cover tension-loaded plate elements connected at their end by transverse partial-joint-penetration groove or fillet welds in which there is more than a single site for the initiation of fatigue cracking, one of which will be more critical than the others depending upon welded joint type and size and material thickness (Frank and Fisher, 1979). Regardless of the site within the joint at which potential crack initiation is considered, the design stress range provided is applicable to connected material at the toe of the weld.

3.4. BOLTS AND THREADED PARTS

The fatigue resistance of bolts subject to tension is predictable in the absence of pretension and prying action; provisions are given for such nonpretensioned details as hanger rods and anchor rods. In the case of pretensioned bolts, deformation of the connected parts through which pretension is applied introduces prying action, the magnitude of which is not completely predictable (Kulak and others, 1987). The effect of prying is not limited to a change in the average axial tension on the bolt but includes bending in the threaded area under the nut. Because of the uncertainties in calculating prying effects, definitive provisions for the design stress range for bolts subject to applied axial tension are not included in this Specification. To limit the uncertainties regarding prying action on the fatigue of pretensioned bolts in details which introduce prying, the design stress range provided in Table A-3.1 is appropriate for extended cyclic loading only if the prying induced by the applied load is small.

Nonpretensioned fasteners are not permitted under this Specification for joints subject to cyclic shear forces. Bolts installed in joints meeting all the requirements for slip-critical connections survive unharmed when subject to cyclic shear stresses sufficient to fracture the connected parts; provisions for such bolts are given in Section 2 of Table A-3.1.

3.5. SPECIAL FABRICATION AND ERECTION REQUIREMENTS

It is essential that when longitudinal backing bars are to be left in place, they be continuous or spliced using flush-ground complete-joint-penetration groove welds before attachment to the parts being joined. Otherwise, the transverse nonfused section constitutes a crack-like defect that can lead to premature fatigue failure or even *brittle fracture* of the built-up member.

In transverse joints subjected to tension a lack-of-fusion plane in T-joints acts as an initial crack-like condition. In groove welds, the root at the backing bar often has discontinuities that can reduce the fatigue resistance of the connection. Removing the backing, back gouging the joint, and rewelding eliminates the undesirable discontinuities.

The addition of contoured fillet welds at transverse complete-joint-penetration groove welds in T- and corner joints and at reentrant corners reduces the stress concentration and improves fatigue resistance.

Experimental studies on welded built-up beams demonstrated that if the surface roughness of flame-cut edges was less than 1000 μin. (25 μm), fatigue cracks would not develop from the flame-cut edge but from the longitudinal fillet welds connecting the beam flanges to the web (Fisher and others, 1970; Fisher and others, 1974). This provides Category B fatigue resistance without the necessity for grinding flame-cut edges.

Reentrant corners at cuts, copes and weld access holes provide a stress concentration point that can reduce fatigue resistance if discontinuities are introduced by punching or thermal cutting. Reaming sub-punched holes and grinding the thermally cut surface to bright metal prevents any significant reduction in fatigue resistance.

The use of run-off tabs at transverse butt-joint groove welds enhances weld soundness at the ends of the joint. Subsequent removal of the tabs and grinding of the ends flush with the edge of the member removes discontinuities that are detrimental to fatigue resistance.

APPENDIX 4

STRUCTURAL DESIGN FOR FIRE CONDITIONS

4.1. GENERAL PROVISIONS

Appendix 4 provides structural engineers with guidance in designing steel-framed building systems and components, including columns, and floor and truss assemblies, for fire conditions. Compliance with the performance objective in Section 4.1.1 can be demonstrated by either structural analysis or component qualification testing.

Thermal expansion and progressive decrease in strength and stiffness are the primary structural responses to elevated temperatures that may occur during fires. An assessment of a design of building components and systems based on structural mechanics that allows designers to address the fire-induced restrained thermal expansions, deformations and material degradation at elevated temperatures can lead to a more robust structural design for fire conditions.

Glossary
Terms pertinent to the design of structural components and systems for fire conditions are presented in the glossary. Terms in common with those in other fire-resistant design documents developed by the SFPE, ICC, NFPA, ASTM and similar organizations are defined in a manner consistent with those documents.

4.1.1. Performance Objective

The performance objective underlying the provisions and guidelines in this Specification is that of life safety. Fire safety levels should depend on the building occupancy, height of building, the presence of active fire mitigation measures, and the effectiveness of fire-fighting. Three limit states exist for elements serving as fire barriers (compartment walls and floors): (1) heat transmission leading to unacceptable rise of temperature on the unexposed surface; (2) breach of barrier due to cracking or loss of integrity; and (3) loss of load-bearing capacity. In general, all three must be considered by the engineer to achieve the desired performance. These three limit states are interrelated in fire-resistant design. For structural elements that are not part of a separating element, the governing limit state is loss of load-bearing capacity.

Specific performance objectives for a facility are determined by the stakeholders in the building process, within the context of the above general performance objective and limit states. In some instances, applicable building codes may stipulate that steel in buildings of certain occupancies and heights be protected by fire-resistant materials or assemblies to achieve specified performance goals.

4.1.4. Load Combinations and Required Strength

Fire safety measures are aimed at three levels: (1) to prevent the outbreak of fires through elimination of ignition sources or hazardous practices; (2) to prevent uncontrolled fire development and flashover through early detection and suppression; and (3) to prevent loss of life or structural collapse through fire protection systems, compartmentation, exit ways, and provision of general structural integrity and other passive measures. Specific structural design provisions to check structural integrity and risk of progressive failure due to severe fires can be developed from principles of structural reliability theory (Ellingwood and Leyendecker, 1978; Ellingwood and Corotis, 1991).

The limit state probability of failure due to fire can be written as

$$P(F) = P(F|D,I)P(D|I)P(I) \qquad \text{(C-A-4-1-1)}$$

where $P[I]$ = probability of ignition, $P[D|I]$ = probability of development of a structurally significant fire, and $P[F|D,I]$ = probability of failure, given the occurrence of the two preceding events. Measures taken to reduce $P(I)$and $P(D|I)$ are mainly nonstructural in nature. Measures taken by the structural engineer to design fire resistance into the structure impact $P(F|D,I)$.

The development of structural design requirements requires a target reliability level, reliability being measured by $P(F)$ in Equation C-A-4-1-1. Analysis of reliability of structural systems for gravity dead and live load (Galambos, Ellingwood, MacGregor, and Cornell, 1982) suggests that the limit state probability of individual steel members and connections is on the order of 10^{-5} to 10^{-4}/year. For redundant steel frame systems, $P(F)$ is on the order of 10^{-6} to 10^{-5}. The *de minimis* risk, that is, the level below which the risk is of regulatory or legal concern and the economic or social benefits of risk reduction are small, is on the order of 10^{-7} to 10^{-6}/year (Pate-Cornell, 1994). If $P(I)$ is on the order of 10^{-4}/year for typical buildings and $P(D|I)$ is on the order of 10^{-2} for office or commercial buildings in urban areas with suppression systems or other protective measures, then $P(F|D,I)$ should be approximately 0.1 to ascertain that the risk due to structural failure caused by fire is socially acceptable.

The use of first-order structural reliability analysis based on this target (conditional) limit state probability leads to the gravity load combination presented as Equation A-4-1. Load combination Equation A-4-1 is the same as Equation C2-3 that appears in Commentary C2.5 of SEI/ASCE 7 (ASCE, 2002), where the probabilistic bases for load combinations for extraordinary events is explained in detail. The factor 0.9 is applied to the dead load when the effect of the dead load is to stabilize the structure; otherwise, the factor 1.2 is applied. The companion action load factors on L and S in that equation reflect the fact that the probability of a coincidence of the peak time-varying load with the occurrence of a fire is negligible (Ellingwood and Corotis, 1991).

Commentary C2.5 of ASCE (2002) contains a second equation that includes 0.2W. That equation is provided so that the stability of the system is checked. The same purpose is accomplished by requiring that the frame be checked under the effect of a small notional lateral load equal to 0.2 percent of story gravity force, acting in combination with the gravity loads. The required strength of the structural component or system designed using these load combinations is on the order of 60 percent to 70 percent of the required strength under full gravity or wind load at normal temperature.

4.2. STRUCTURAL DESIGN FOR FIRE CONDITIONS BY ANALYSIS

4.2.1. Design-Basis Fire

Once a fuel load has been agreed upon for the occupancy, the designer should demonstrate the effect of various fires on the structure by assessing the temperature-time relationships for various ventilation factors. These relations may result in different structural responses, and it is useful to demonstrate the capability of the structure to withstand such exposures. The effects of a localized fire should also be assessed to ascertain that local damage is not excessive. Based on these results, connections and edge details can be specified to provide a structure that is sufficiently robust.

4.2.1.1. Localized Fire

Localized fires may occur in large open spaces, such as the pedestrian area of covered malls, concourses of airport terminals, warehouses, and factories, where fuel packages are separated by large aisles or open spaces. In such cases, the radiant heat flux can be estimated by a point source approximation, requiring the heat release rate of the fire and separation distance between the center of the fuel package and the closest surface of the steelwork. The heat release rate can be determined from experimental results or may be estimated if the mass loss rate per unit floor area occupied by the fuel is known. Otherwise, a steady-state fire may be assumed.

4.2.1.2. Post-Flashover Compartment Fires

Caution should be exercised when determining temperature-time profiles for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with an open (or exposed) floor area in excess of 5,000 ft^2 (465 m^2). In such cases, it is unlikely that all combustibles will burn in the space simultaneously. Instead, burning will be most intense in, or perhaps limited to, the combustibles nearest to a ventilation source. For modest-sized compartments with low aspect ratios, the temperature history of the design fire can be determined by algebraic equations or computer models, such as those described in the SFPE *Handbook of Fire Protection Engineering* (SFPE, 2002).

4.2.1.3. Exterior Fires

A design guide is available for determining the exposure resulting from an exterior fire (AISI, 1979).

4.2.1.4. Fire Duration

Caution should be exercised when determining the fire duration for spaces with high aspect ratios, for example, 5:1 or greater, or for large spaces, for example, those with a floor area in excess of 5,000 ft^2 (465 m^2). The principal difficulty lies in obtaining a realistic estimate for the mass loss rate, given that all combustibles within the space may not be burning simultaneously. Failure to recognize uneven burning will result in an overestimation of the mass burning rate and an underestimation of the fire duration by a significant margin. Note: some computation methods may implicitly determine the duration of the fire, in which case the calculation of mass loss rate is unnecessary.

Where a parametric curve is used to define a post-flashover fire, the duration is determined by means of the fuel versus ventilation provisions, not explicitly by loss of mass. This clause should not limit the use of temperature-time relationships to those where duration is calculated, as stated above, as these tend to be localized fires and external fire.

4.2.1.5. Active Fire Protection Systems

Due consideration should be given to the reliability and effectiveness of active fire protection systems when describing the design-basis fire. When an automatic sprinkler system is installed, the total fuel load may be reduced by up to 60 percent (Eurocode 1, 1991). The maximum reduction in the fuel load should be considered only when the automatic sprinkler system is considered to be of the highest reliability, for example, reliable and adequate water supply, supervision of control valves, regular schedule for maintenance of the automatic sprinkler system developed in accordance with NFPA (2002), or alterations of the automatic sprinkler system are considered any time alterations for the space are considered.

For spaces with automatic smoke and heat vents, computer models are available to determine the smoke temperature (SFPE, 2002). Reduction in the temperature profile as a result of smoke and heat vents should only be considered for reliable installations of smoke and heat vents. As such, a regular maintenance schedule for the vents needs to be established in accordance with NFPA (2002a).

4.2.2. Temperatures in Structural Systems under Fire Conditions

The heat transfer analysis may range from one-dimensional analyses where the steel is assumed to be at uniform temperature to three-dimensional analyses. The uniform temperature assumption is appropriate in a "lumped heat capacity analysis" where a steel column, beam or truss element is uniformly heated along the entire length and around the entire perimeter of the exposed section and the protection system is uniform along the entire length and around the entire perimeter of the section. In cases with nonuniform heating or where different protection methods are used on different sides of the column, a one-dimensional analysis should be conducted for steel column assemblies. Two-dimensional analyses are appropriate for beams, bar joists or truss elements supporting floor or roof slabs.

Heat transfer analyses should consider changes in material properties with increasing temperature for all materials included in the assembly. This may be done in the lumped heat capacity analysis using an effective property value, determined at a temperature near the estimated mid-point of the temperature range expected to be experienced by that component over the duration of the exposure. In the one- and two-dimensional analyses, the variation in properties with temperature should be explicitly included.

The boundary conditions for the heat transfer analysis shall consider radiation heat transfer in all cases and convection heat transfer if the exposed element is submerged in the smoke or is being subjected to flame impingement. The presence of fire resistive materials in the form of insulation, heat screens or other protective measures shall be taken into account, if appropriate.

Lumped Heat Capacity Analysis. This first-order analysis to predict the temperature rise of steel structural members can be conducted using algebraic equations iteratively. This approach assumes that the steel member has a uniform temperature, applicable to cases where the steel member is unprotected or uniformly protected (on all sides), and is exposed to fire around the entire perimeter of the assembly containing the steel member. Caution should be used when applying this method to steel beams supporting floor and roof slabs, as the approach will overestimate the temperature rise in the beam. In addition, where this analysis is used as input for the structural analysis of a fire-exposed, steel beam supporting a floor and roof slab, the thermally induced moments will not be simulated as a result of the uniform temperature assumption.

Unprotected steel members. The temperature rise in an unprotected steel section in a short time period shall be determined by

$$\Delta T_s = \frac{a}{c_s \left(\dfrac{W}{D}\right)} (T_F - T_s)\,\Delta t \qquad \text{(C-A-4-2-1)}$$

The heat transfer coefficient, a, is determined from

$$a = a_c + a_r \qquad \text{(C-A-4-2-2)}$$

where

a_c = convective heat transfer coefficient

a_r = radiative heat transfer coefficient, given as

$$a_r = \frac{5.67 \times 10^{-8} \varepsilon_F}{T_F - T_S}\left(T_F^4 - T_s^4\right)$$

For the standard exposure, the convective heat transfer coefficient, a_c, can be approximated as 25 W/m^2-°C. The parameter, ε_F, accounts for the emissivity

TABLE C-A-4-2.1
Guidelines for Estimating ε_F

Type of Assembly	ε_F
Column, exposed on all sides	0.7
Floor beam: Imbedded in concrete floor slab, with only bottom flange of beam exposed to fire	0.5
Floor beam, with concrete slab resting on top flange of beam	
Flange width : beam depth ratio $\geq$ 0.5	0.5
Flange width : beam depth ratio $<$ 0.5	0.7
Box girder and lattice girder	0.7

of the fire and the view factor. Estimates for ε_F, are suggested in Table C-A-4-2.1.

For accuracy reasons, a maximum limit for the time step, Δt, is suggested as 5 sec.

The fire temperature needs to be determined based on the results of the design fire analysis. As alternatives, the standard time-temperature curves indicated in ASTM E119 (ASTM, 2000) for building fires or ASTM E1529 (ASTM, 2000a) for petrochemical fires may be selected.

Protected Steel Members. This method is most applicable for steel members with contour protection schemes, in other words, where the insulating or (protection) material follows the shape of the section. Application of this method for box protection methods will generally result in the temperature rise being overestimated. The approach assumes that the outside insulation temperature is approximately equal to the fire temperature. Alternatively, a more complex analysis may be conducted which determines the exterior insulation temperature from a heat transfer analysis between the assembly and the exposing fire environment.

If the thermal capacity of the insulation is much less than that for the steel, such that the following inequality is satisfied:

$$c_s W/D > 2d_p \rho_p c_p \qquad \text{(C-A-4-2-3)}$$

Then, Equation C-A-4-2-4 can be applied to determine the temperature rise in the steel:

$$\Delta T_s = \frac{k_p}{c_s d_p \dfrac{W}{D}} (T_F - T_s) \Delta t \qquad \text{(C-A-4-2-4)}$$

If the thermal capacity of the insulation needs to be considered (such that the inequality in Equation C-A-4-2-3 is not satisfied), then Equation C-A-4-2-5 should be applied:

$$\Delta T_s = \frac{k_p}{d_p} \left[\frac{T_F - T_s}{c_s \dfrac{W}{D} + \dfrac{c_p \rho_p d_p}{2}} \right] \Delta t \qquad \text{(C-A-4-2-5)}$$

The maximum limit for the time step, Δt, should be 5 sec.

Ideally, material properties should be considered as a function of temperature. Alternatively, material properties may be evaluated at a mid-range temperature expected for that component. For protected steel members, the material properties may be evaluated at 300 °C, and for protection materials, a temperature of 500 °C may be considered.

External Steelwork. Temperature rise can be determined by applying the following equation:

$$\Delta T_s = \frac{q''}{c_s\left(\dfrac{W}{D}\right)}\Delta t \tag{C-A-4-2-6}$$

where q'' is the net heat flux incident on the steel member

Advanced Calculation Methods. The thermal response of steel members may be assessed by application of a computer model. A computer model for analyzing the thermal response of the steel members should consider the following:

- Exposure conditions established based on the definition of a design fire. The exposure conditions need to be stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux. The incident heat flux is dependent on the design fire scenario and the location of the structural assembly. The heat flux emitted by the fire or smoke can be determined from a fire hazard analysis. Exposure conditions are established based on the definition of a design fire. The exposure conditions are stipulated either in terms of a time-temperature history, along with radiation and convection heat transfer parameters associated with the exposure, or as an incident heat flux.
- Temperature-dependent material properties.
- Temperature variation within the steel member and any protection components, especially where the exposure varies from side to side.

Nomenclature:

A_m	surface area of a member per unit length, ft (m)
A_p	area of the inner surface of the fire protection material per unit length of the member, ft (m)
A_c	cross-sectional area, in.^{2} (m^2)
D	heat perimeter, in. (m)
T	temperature, °F (°C)
V	volume of a member per unit length, in.^{2} (m^2)
W	weight (mass) per unit length, lb/ft (kg/m)
a	heat transfer coefficient, Btu/ft^2·sec·°F (W/m^2·°C)
c	specific heat, Btu/lb·°F (J/kg·°C)
d	thickness, in. (m)

$h_{net,d}$	design value of the net heat flux per unit area, Btu/sec·ft^2 (W/m^2)
k	thermal conductivity, Btu/ft·sec·°F (W/m·°C)
l	length, ft (m)
t	time in fire exposure, seconds
Δt	time interval, seconds
ρ	density, lb/ft^3 (kg/m^3)

Subscripts:

a	steel
c	convection
m	member
p	fire protection material
r	radiation
s	steel
t	dependent on time
T	dependent on temperature

4.2.3. Material Strengths at Elevated Temperatures

The properties for steel and concrete at elevated temperatures are adopted from the ECCS *Model Code on Fire Engineering* (ECCS, 2001), Section III.2, "Material Properties." These generic properties are consistent with those in Eurocodes 3 (Eurocode 3, 2002) and 4 (Eurocode 4, 2003), and reflect the consensus of the international fire engineering and research community. The background information for the mechanical properties of structural steel at elevated temperatures can be found in Cooke (1988) and Kirby and Preston (1988).

4.2.4. Structural Design Requirements

The resistance of the structural system in the design basis fire may be determined by:

(a) Structural analysis of individual elements where the effects of restraint to thermal expansion and bowing may be ignored but the reduction in strength and stiffness with increasing temperature is incorporated.
(b) Structural analysis of assemblies/subframes where the effects of restrained thermal expansion and thermal bowing are considered by incorporating geometric and material nonlinearities.
(c) Global structural analysis where restrained thermal expansion, thermal bowing, material degradation and geometric nonlinearity are considered.

4.2.4.1. General Structural Integrity

The requirement for general structural integrity is consistent with that appearing in Section 1.4 of ASCE (2002). Structural integrity is the ability of the structural system to absorb and contain local damage or failure without developing into a progressive collapse that involves the entire structure or a disproportionately large part of it.

The Commentary C1.4 to Section 1.4 of ASCE (2002) contains guidelines for the provision of general structural integrity. Compartmentation (subdivision of buildings/stories in a building) is an effective means of achieving resistance to progressive collapse as well as preventing fire spread, as a cellular arrangement of structural components that are well tied together provides stability and integrity to the structural system as well as insulation.

4.2.4.2. Strength Requirements and Deformation Limits

As structural elements are heated, their expansion is restrained by adjacent element and connections. Material properties degrade with increasing temperature. Load transfer can occur from hotter elements to adjacent cooler elements. Excessive deformation may be of benefit in a fire as it allows release of thermally induced stresses. Deformation is acceptable once horizontal and vertical separation as well as the overall load bearing capacity of the structural system is maintained.

4.2.4.3. Methods of Analysis

4.2.4.3a. Advanced Methods of Analysis

Advanced methods are required when the overall structural system response to fire, the interaction between structural members and separating elements in fire, or the residual strength of the structural system following a fire must be considered.

4.2.4.3b. Simple Methods of Analysis

Simple methods may suffice when a structural member or component can be assumed to be subjected to uniform heat flux on all sides and the assumption of a uniform temperature is reasonable as, for example, in a free-standing column.

4.2.4.4. Design Strength

The design strength for structural steel members and connections is calculated as ϕR_n, in which $R_n =$ nominal strength, in which the deterioration in strength at elevated temperature is taken into account, and ϕ is the resistance factor. The nominal strength is computed as in Chapters C, D, E, F, G, H, I, J and K of the Specification, using material strength and stiffnesses at elevated temperatures defined in Tables A-4.2.1 and A-4.2.2. While ECCS (2001) and Eurocode 1 (1991) specify partial material factors as equal to 1.0 for "accidental" limit states, the uncertainties in strength at elevated temperatures are substantial and in some cases are unknown. Accordingly, the resistance factors herein are the same as those at ordinary conditions.

4.3. DESIGN BY QUALIFICATION TESTING

Qualification testing is an acceptable alternative to design by analysis for providing fire resistance. It is anticipated that the basis will be ASCE (1998), ASTM (2000) and similar documents.

An unrestrained condition is one in which expansion at the support of a load carrying element is not resisted by forces external to the element and the supported ends are free to expand and rotate. A steel member bearing on a wall in a single span or at the end span of multiple spans should be considered unrestrained when the wall has not been designed and detailed to resist thermal thrust.

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APPENDIX 5

EVALUATION OF EXISTING STRUCTURES

5.1. GENERAL PROVISIONS

The load combinations referred to in this chapter pertain to gravity loading because it is the most prevalent condition encountered. If other loading conditions are a consideration, such as lateral loads, the appropriate load combination from ASCE (2002) or from the applicable building code should be used. The engineer of record for a project is generally established by the owner.

5.2. MATERIAL PROPERTIES

1. Determination of Required Tests

The extent of tests required depends on the nature of the project, the criticality of the structural system or member evaluated, and the availability of records pertinent to the project. Thus, the engineer of record has the responsibility to determine the specific tests required and the locations from which specimens are to be obtained.

2. Tensile Properties

Samples required for tensile tests should be removed from regions of reduced stress, such as at flange tips at beam ends and external plate edges, to minimize the effects of the reduced area. The number of tests required will depend on whether they are conducted to merely confirm the strength of a known material or to establish the strength of some other steel.

It should be recognized that the yield stress determined by standard ASTM methods and reported by mills and testing laboratories is somewhat greater than the static yield stress because of dynamic effects of testing. Also, the test specimen location may have an effect. These effects have already been accounted for in the nominal strength equations in the Specification. However, when strength evaluation is done by load testing, this effect should be accounted for in test planning because yielding will tend to occur earlier than otherwise anticipated. The static yield stress, F_{ys}, can be estimated from that determined by routine application of ASTM methods, F_y, by the following equation (Galambos, 1978; Galambos, 1998):

$$F_{ys} = R(F_y - 4) \qquad \text{(C-A-5-2-1)}$$

$$[\text{S.I.}: F_{ys} = R(F_y - 27)] \qquad \text{(C-A-5-2-1M)}$$

where

F_{ys} = static yield stress, ksi (MPa)
F_y = reported yield stress, ksi (MPa)
R = 0.95 for tests taken from web specimens
= 1.00 for tests taken from flange specimens

The R factor in Equation C-A-5-2-1 accounts for the effect of the coupon location on the reported yield stress. Prior to 1997, certified mill test reports for structural shapes were based on specimens removed from the web, in accordance with ASTM A6/A6M (ASTM, 2003). Subsequently the specified coupon location was changed to the flange. During 1997–1998, there was a transition from web specimens to flange specimens as the new provisions of ASTM A6/A6M (ASTM, 2003) were adopted.

4. Base Metal Notch Toughness

The engineer of record shall specify the location of samples. Samples shall be cored, flame cut or saw cut. The engineer of record will determine if remedial actions are required, such as the possible use of bolted splice plates.

5. Weld Metal

Because connections typically are more reliable than structural members, strength testing of weld metal is not usually necessary. However, field investigations have sometimes indicated that complete-joint-penetration groove welds, such as at beam-to-column connections, were not made in accordance with AWS D1.1 (AWS, 2004). The specified provisions in AWS D1.1, Section 5.2.4 provide a means for judging the quality of such a weld. Where feasible, any samples removed should be obtained from compression splices rather than tension splices, because the effects of repairs to restore the sampled area are less critical.

6. Bolts and Rivets

Because connections typically are more reliable than structural members, removal and strength testing of fasteners is not usually necessary. However, strength testing of bolts is required where they can not be properly identified otherwise. Because removal and testing of rivets is difficult, assuming the lowest rivet strength grade simplifies the investigation.

5.3. EVALUATION BY STRUCTURAL ANALYSIS

2. Strength Evaluation

Resistance and safety factors reflect variations in determining strength of members and connections, such as uncertainty in theory and variations in material properties and dimensions. If an investigation of an existing structure indicates that there are variations in material properties or dimensions significantly greater than those anticipated in new construction, the engineer of record should consider the use of more conservative values.

5.4. EVALUATION BY LOAD TESTS

1. Determination of Live Load Rating by Testing

Generally, structures that can be designed according to the provisions of this Specification need no confirmation of calculated results by test. However, special situations may arise when it is desirable to confirm by tests the results of calculations. Minimal test procedures are provided to determine the live load rating of a structure. However, in no case is the live load rating determined by test to exceed that which can be calculated using the provisions of this Specification. This is not intended to preclude testing to evaluate special conditions or configurations that are not adequately covered by this Specification.

It is essential that the engineer of record take all necessary precautions to ascertain that the structure does not fail catastrophically during testing. A careful assessment of structural conditions before testing is a fundamental requirement. This includes accurate measurement and characterization of the size and strength of members, connections and details. All safety regulations of OSHA and other pertinent bodies must be strictly adhered to. Shoring and scaffolding should be used as required in the proximity of the test area to mitigate against unexpected circumstances. Deformations must be carefully monitored and structural conditions must be continually evaluated. In some cases it may be desirable to monitor strains as well.

The engineer of record must use judgment to determine when deflections are becoming excessive and terminate the tests at a safe level even if the desired loading has not been achieved. Incremental loading is specified so that deformations can be accurately monitored and the performance of the structure carefully observed. Load increments should be small enough initially so that the onset of significant yielding can be determined. The increment can be reduced as the level of inelastic behavior increases, and the behavior at this level carefully evaluated to determine when to safely terminate the test. Periodic unloading after the onset of inelastic behavior will help the engineer of record determine when to terminate the test to avoid excessive permanent deformation or catastrophic failure.

It must be recognized that the margin of safety at the maximum load level used in the test may be very small, depending on such factors as the original design, the purpose of the tests, and the condition of the structure. Thus, it is imperative that all appropriate safety measures be adopted. It is recommended that the maximum live load used for load tests be selected conservatively. It should be noted that experience in testing more than one bay of a structure is limited.

The provision limiting increases in deformations for a period of one hour is given so as to have positive means that the structure is stable at the loads evaluated.

2. Serviceability Evaluation

In certain cases serviceability performance must be determined by load testing. It should be recognized that complete recovery (in other words, return to initial deflected shape) after removal of maximum load is unlikely because of phenomena such as local yielding, slip at the slab interface in composite construction, creep in concrete slabs, localized crushing or deformation at shear connections in slabs, slip in bolted connections, and effects of continuity. Because most structures exhibit some slack when load is first applied, it is appropriate to project the load-deformation curve back to zero load to determine the slack and exclude it from the recorded deformations. Where desirable, the applied load sequence can be repeated to demonstrate that the structure is essentially elastic under service loads and that the permanent set is not detrimental.

5.5. EVALUATION REPORT

Extensive evaluation and load testing of existing structures is often performed when appropriate documentation no longer exists or when there is considerable disagreement about the condition of a structure. The resulting evaluation is only effective if well documented, particularly when load testing is involved. Furthermore, as time passes, various interpretations of the results can arise unless all parameters of the structural performance, including material properties, strength, and stiffness, are well documented.

APPENDIX 6

STABILITY BRACING FOR COLUMNS AND BEAMS

6.1. GENERAL PROVISIONS

The design requirements of Appendix 6 consider two general types of bracing systems, relative and nodal, as shown in Figure C-A-6.1.

A relative column brace system (such as diagonal bracing or shear walls) is attached to two locations along the length of the column that defines the unbraced length. The relative brace system shown consists of the diagonal and the strut that controls the movement at one end of the unbraced length, A, with respect to the other end of the unbraced length, B. The diagonal and the strut both contribute to the strength and stiffness of the relative brace system. However, when the strut is a floor beam, its stiffness is large compared to the diagonal so the diagonal controls the strength and stiffness of the relative brace.

A nodal brace controls the movement only at the particular brace point, without direct interaction with adjacent braced points. Therefore to define an unbraced length, there must be additional adjacent brace points as shown in Figure C-A-6.1. The two nodal column braces at C and D that are attached to the rigid abutment define the unbraced length for which $K = 1.0$ can be used. For beams a cross frame between two adjacent beams at midspan is a nodal brace because it prevents twist of the beams only at the particular cross frame location. The unbraced length is half the span length. The twist at the ends of the two beams is prevented by the beam-to-column connections at the end supports. Similarly, a nodal lateral brace attached at midspan to the top flange of the beams and a rigid support assumes that there is no lateral movement at the column locations.

The brace requirements are intended to enable a member to potentially reach a maximum load based on the unbraced length between the brace points and $K = 1.0$. This is not the same as the no-sway buckling load as illustrated in Figure C-A-6.2 for a braced cantilever. The critical stiffness is 1.0 P_e/L, corresponding to $K = 1.0$. A brace with five times this stiffness is necessary to reach 95 percent of the $K = 0.7$ limit. Theoretically, an infinitely stiff brace is required to reach the no-sway limit. Bracing required to reach specified rotation capacities or ductility limits is beyond the scope of these recommendations. Member inelasticity has no significant effect on the brace requirements (Yura, 1995).

Winter developed the concept of a dual requirement for bracing design: strength and stiffness (Winter, 1958; Winter, 1960). The brace force is a function of the initial column out-of-straightness, Δ_o, and the brace stiffness, β. For a relative brace system, the relationship between column load, brace stiffness and sway displacement is shown in Figure C-A-6.3. If $\beta = \beta_i$, the critical brace stiffness for a perfectly plumb member, then $P = P_e$ only if the sway deflection gets very large. Unfortunately, such large displacements produce large brace forces. For practical design, Δ must be kept small at the factored load level.

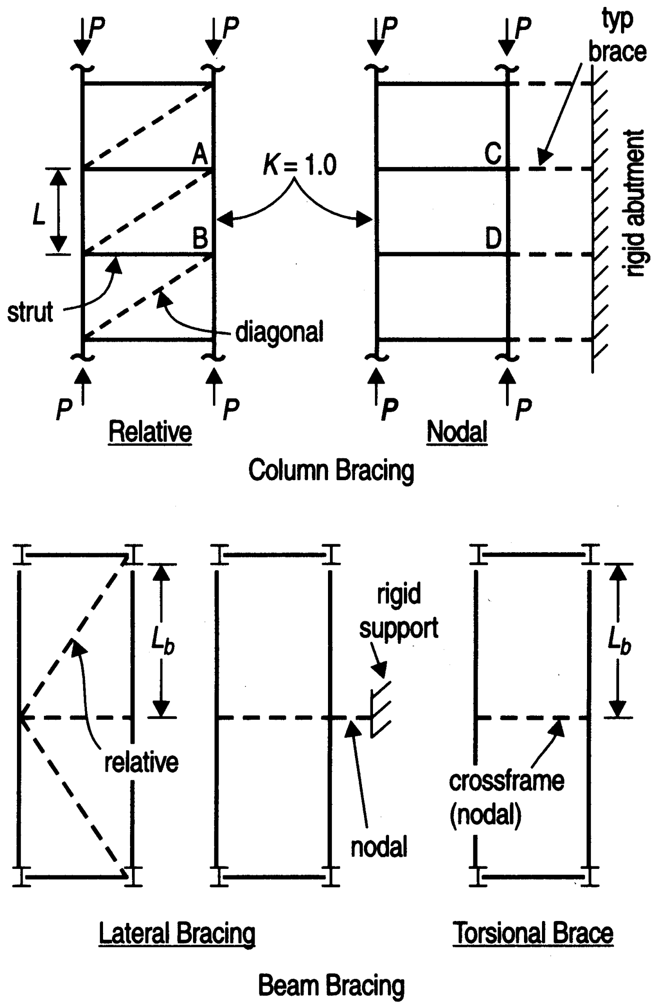


Fig. C-A-6.1. Types of bracing.

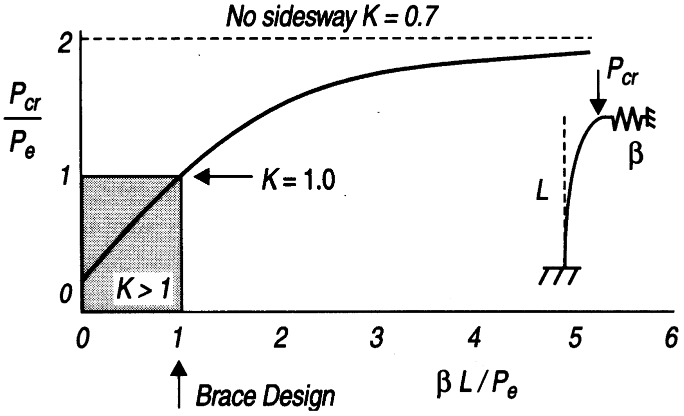


Fig. C-A-6.2. Braced cantilever.

The brace stiffness requirements, β_{br}, for frames, columns, and beams were chosen as twice the critical stiffness. All brace stiffness requirements use a $\phi = 0.75$. For the relative brace system shown in Figure C-A-6.3, $\beta_{br} = 2\beta_i$ gives $P_{br} = 0.4\% P_e$ for $\Delta_o = 0.002L$. If the brace stiffness provided, β_{act}, is different from the requirement, then the brace force or brace moment can be multiplied by the following factor:

$$\frac{1}{2 - \dfrac{\beta_{br}}{\beta_{act}}} \tag{C-A-6-1}$$

No ϕ is specified in the brace strength requirements since ϕ is included in the component design strength provisions in other chapters of this Specification.

The initial displacement, Δ_o, for relative and nodal braces is defined with respect to the distance between adjacent braces, as shown in Figure C-A-6.4. The initial Δ_o is a displacement from the straight position at the brace points caused by sources other than brace elongations from gravity loads or compressive forces, such as displacements caused by wind or other lateral forces, erection tolerances, column

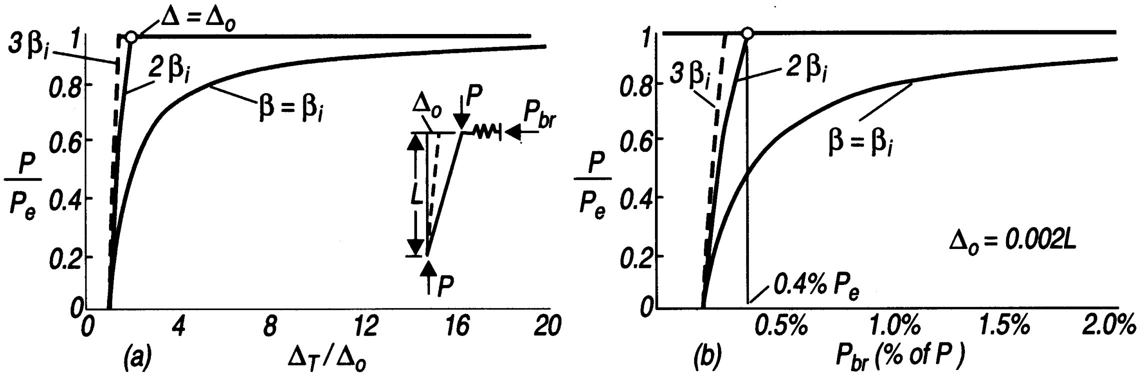


Fig. C-A-6.3. Effect of initial out-of-plumbness.

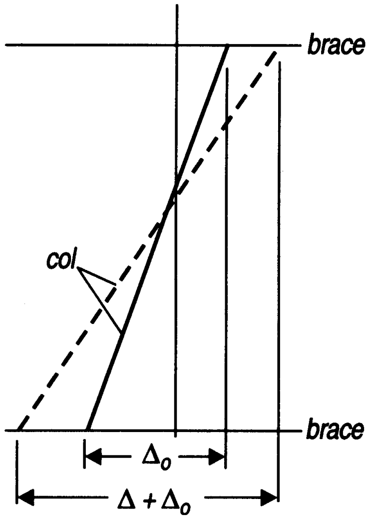


Fig. C-A-6.4. Definitions of initial displacements for relative and nodal braces.

shortening, etc. The brace force recommendations for frames, columns and beam lateral bracing are based on an assumed $\Delta_o = 0.002L$, where L is the distance between adjacent brace points. For torsional bracing of beams, an initial twist angle, θ_o, is assumed where $\theta_o = 0.002L/h_o$, and h_o is the distance between flange centroids. For other Δ_o and θ_o values, use direct proportion to modify the brace strength requirements, P_{br} and M_{br}. For cases where it is unlikely that all columns in a story are out-of-plumb in the same direction, Chen and Tong recommend an average $\Delta_o = 0.002L/\sqrt{n_o}$ where n_o columns, each with a random Δ_o, are to be stabilized by the brace system (Chen and Tong, 1994). This reduced Δ_o would be appropriate when combining the stability brace forces with wind and seismic forces.

Brace connections, if they are flexible or can slip, should be considered in the evaluation of the bracing stiffness as follows:

$$\frac{1}{\beta_{act}} = \frac{1}{\beta_{conn}} + \frac{1}{\beta_{brace}} \qquad \text{(C-A-6-2)}$$

The brace system stiffness, β_{act}, is less than the smaller of the connection stiffness, β_{conn}, or the stiffness of the brace, β_{brace}. Slip in connections with standard holes need not be considered except when only a few bolts are used. When evaluating the bracing of rows of columns or beams, consideration must be given to the accumulation of the brace forces along the length of the brace that results in a different displacement at each beam or column location. In general, brace forces can be minimized by increasing the number of braced bays and using stiff braces.

6.2. COLUMNS

For nodal column bracing, the critical stiffness is a function of the number of intermediate braces (Winter, 1958; Winter, 1960). For one intermediate brace, $\beta_i = 2P/L_b$, and for many braces $\beta_i = 4P/L_b$. The relationship between the critical stiffness and the number of braces, n, can be approximated (Yura, 1995) as $\beta_i = N_i P/L_b$, where $N_i = 4 - 2/n$. The most severe case (many braces) was adopted for the brace stiffness requirement, $\beta_{br} = 2 \times 4P/L_b$. The brace stiffness, Equation A-6-4, can be reduced by the ratio, $N_i/4$, to account for the actual number of braces.

The unbraced length, L_b, in Equation A-6-4 is assumed to be equal to the length L_q that enables the column to reach P_u. When the actual bracing spacing is less than L_q, the calculated required stiffness may become quite conservative since the stiffness equations are inversely proportional to L_b. In such cases, L_q can be substituted for L_b. (This substitution is also applicable for the beam nodal bracing formulations given in Equations A-6-8 and A-6-9.) For example, a W12×53 (W310×79) with $P_u = 400$ kips (1 780 kN) can have a maximum unbraced length of 14 ft (4.3 m) for A36 (A36M) steel. If the actual bracing spacing is 8 ft (2.4 m), then 14 ft (4.3 m) may be used in Equation A-6-4 to determine the required stiffness. The use of L_q in Equation A-6-4 provides reasonable estimates of the brace stiffness requirements; however, the solution can still result in conservative estimates of the stiffness requirements. Improved accuracy can be obtained by treating the system as a continuous bracing system as discussed in Galambos (1998) and Lutz and Fisher (1985).

With regards to the brace strength requirements, Winter's rigid model only accounts for force effects from lateral displacements and would derive a brace force of 0.8 percent P_u, which accounts only for lateral displacement force effects. To account for the additional force due to member curvature, this theoretical force has been increased to $1.0\% P_u$.

6.3. BEAMS

Beam bracing must prevent twist of the section, not lateral displacement. Both lateral bracing (for example, joists attached to the compression flange of a simply supported beam) and torsional bracing (for example, a cross frame or diaphragm between adjacent girders) can effectively control twist. Lateral bracing systems that are attached near the beam centroid are ineffective. For beams with double curvature, the inflection point can not be considered a brace point because twist occurs at that point (Galambos, 1998). A lateral brace on one flange near the inflection point also is ineffective. In double curvature cases the lateral brace near the inflection point must be attached to both flanges to prevent twist, or torsional bracing must be used. The beam brace requirements are based on the recommendations in Yura (1993).

1. Lateral Bracing

For lateral bracing, the following stiffness requirement was derived following Winter's approach:

$$\beta_{br} = 2N_i(C_b P_f)\, C_t C_d / \phi L_b \qquad \text{(C-A-6-3)}$$

where

N_i = 1.0 for relative bracing
= $(4-2/n)$ for discrete bracing
n = number of intermediate braces
P_f = beam compressive flange force
= $\pi^2 E I_{yc} / L_b{}^2$
I_{yc} = out-of-plane moment of inertia of the compression flange
C_b = moment modifier from Chapter F
C_t = accounts for top flange loading (use $C_t = 1.0$ for centroidal loading)
= $1 + (1.2/n)$
C_d = double curvature factor (compression in both flanges)
= $1 + (M_S/M_L)^2$
M_S = smallest moment causing compression in each flange
M_L = largest moment causing compression in each flange

The C_d factor varies between 1.0 and 2.0 and is applied only to the brace closest to the inflection point. The term $(2N_i C_t)$ can be conservatively approximated as

10 for any number of nodal braces and 4 for relative bracing and $(C_b P_f)$ can be approximated by M_u/h which simplifies Equation C-A-6-3 to the stiffness requirements given by Equations A-6-6 and A-6-8. Equation C-A-6-3 can be used in lieu of Equations A-6-6 and A-6-8.

The brace strength requirement for relative bracing is

$$P_{br} = 0.004 M_u C_t C_d / h_o \tag{C-A-6-4a}$$

and for nodal bracing

$$P_{br} = 0.01 M_u C_t C_d / h_o \tag{C-A-6-4b}$$

They are based on an assumed initial lateral displacement of the compression flange of $0.002L_b$. The brace strength requirements of Equations A-6-5 and A-6-7 are derived from Equations C-A-6-4a and C-A-6-4b assuming top flange loading ($C_t = 2$). Equations C-A-6-4a and C-A-6-4b can be used in lieu of Equations A-6-5 and A-6-7, respectively.

2. Torsional Bracing

Torsional bracing can either be attached continuously along the length of the beam (for example, metal deck or slabs) or be located at discrete points along the length of the member (for example, cross frames). With respect to the girder response, torsional bracing attached to the tension flange is just as effective as a brace attached at mid-depth or to the compression flange. Although the girder response is generally not sensitive to the brace location, the position of the brace on the cross section does have an effect on the stiffness of the brace itself. For example, a torsional brace attached on the bottom flange will often bend in single curvature (for example, with a flexural stiffness of $2EI/L$ based on the brace properties), while a brace attached on the top flange will often bend in reverse curvature (for example, with a flexural stiffness of $6EI/L$ based on the brace properties). Partially restrained connections can be used if their stiffness is considered in evaluating the torsional brace stiffness.

The torsional brace requirements are based on the buckling strength of a beam with a continuous torsional brace along its length presented in Taylor and Ojalvo (1966) and modified for cross-section distortion in Yura (1993).

$$M_u \le M_{cr} = \sqrt{(C_{bu} M_o)^2 + \frac{C_b^2 E I_y \overline{\beta}_T}{2C_{tt}}} \tag{C-A-6-5}$$

The term $(C_{bu} M_o)$ is the buckling strength of the beam without torsional bracing. $C_{tt} = 1.2$ when there is top flange loading and $C_{tt} = 1.0$ for centroidal loading. $\overline{\beta}_T = n\beta_T/L$ is the continuous torsional brace stiffness per unit length or its equivalent when n nodal braces, each with a stiffness β_T, are used along the span L and the 2 accounts for initial out-of-straightness. Neglecting the unbraced beam buckling term gives a conservative estimate of the torsional brace stiffness requirement (Equation A-6-11).

The strength requirements for beam torsional bracing were developed based upon an assumed initial twist imperfection of $\theta_o = 0.002L_b/h_o$, where h_o is equal to the depth of the beam. Providing at least twice the ideal stiffness results in a brace force, $M_{br} = \beta_T\theta_o$. Using the LRFD formulation of Equation A-6-11 (without ϕ), the strength requirement for the torsional bracing is

$$M_{br} = \beta_T\theta_o = \frac{2.4LM_u^2}{nEI_yC_b^2}\frac{L_b}{500h_o} \tag{C-A-6-6}$$

To obtain Equation A-6-9, the equation was simplified as follows:

$$M_{br} = \frac{2.4LM_u^2}{nEI_yC_b^2}\frac{L_b}{500h_o}\frac{\pi^2L_b^2}{\pi^2L_b^2} = \frac{2.4\pi^2M_uL}{500nL_bC_b^2}\frac{M_u}{h_o}\frac{L_b^2}{C_b\pi^2EI_y} \tag{C-A-6-7}$$

The term M_u/h_o can be approximated as the flange force, P_f, and the term $L_b^2/C_b\pi^2EI_y$ can be represented as the reciprocal of twice the buckling strength of the flange ($^1/_2P_f$). Substituting for these terms and evaluating the constants results in

$$M_{br} = \frac{0.024M_uL}{nC_bL_b} \tag{C-A-6-8}$$

which is the expression given in Equation A-6-9.

Equations A-6-9 and A-6-12 give the strength and stiffness requirements for doubly symmetric beams. For singly symmetric sections these equations will generally be conservative. Better estimates of the strength requirements for torsional bracing of singly symmetric sections can be obtained with Equation C-A-6-6 by replacing I_y with I_{eff} as given in the following expression:

$$I_{eff} = I_{yc} + \frac{t}{c}I_{yt} \tag{C-A-6-9}$$

where t is the distance from the neutral axis to the extreme tensile fibers, c is the distance from the neutral axis to the extreme compressive fibers, and I_{yc} and I_{yt} are the respective moments of inertia of compression and tension flanges about an axis through the web. Good estimates of the stiffness requirements of torsional braces for singly symmetric I-shaped beams may be obtained using Equation A-6-11 and replacing I_y with I_{eff} given in Equation C-A-6-9.

The β_{sec} term in Equations A-6-10, A-6-12 and A-6-13 accounts for cross-section distortion. A web stiffener at the brace point reduces cross-sectional distortion and improves the effectiveness of a torsional brace. When a cross frame is attached near both flanges or a diaphragm is approximately the same depth as the girder, then web distortion will be insignificant so β_{sec} equals infinity. The required bracing stiffness, β_{Tb}, given by Equation A-6-10 was obtained by solving the following expression that represents the brace system stiffness including distortion effects:

$$\frac{1}{\beta_T} = \frac{1}{\beta_{Tb}} + \frac{1}{\beta_{sec}} \tag{C-A-6-10}$$

Parallel chord trusses with both chords extended to the end of the span and attached to supports can be treated like beams. In Equations A-6-5 through A-6-9, M_u may be taken as the maximum compressive chord force times the depth of the truss to determine the brace strength and stiffness requirements. Cross-section distortion effects, β_{sec}, need not be considered when full-depth cross frames are used for bracing. When either chord does not extend to the end of the span, consideration should be given to control twist near the ends of the span by the use of cross frames or ties.

APPENDIX 7

DIRECT ANALYSIS METHOD

Appendix 7, the direct analysis method, addresses a new method for the stability analysis and design of structural steel systems comprised of moment frames, braced frames, shear walls or combinations thereof (AISC-SSRC, 2003a). While the precise formulation of the method is unique to the AISC Specification, some of its features have similarities to other major design specifications around the world including the Eurocodes, the Australian Standard, the Canadian Standard and ACI 318.

The direct analysis method has been developed with the goal of more accurately determining the load effects in the structure in the analysis stage and eliminating the need for calculating the effective buckling length (K factor) for columns in the first term of the beam-column interaction equations. This method is, therefore, a major step forward in the design of steel moment frames from past editions of the Specification. In addition, the method can be used for the design of braced frames and combined frame systems. Thus, this one method can be used for the design of all types of steel framed structures used in practice. The method can be expanded in the future beyond its use as a second-order elastic analysis tool as presented here. For example, it can be applied with inelastic or plastic analysis. Also, it can be used in the analysis of composite structures, although this application is not explicitly addressed in this Specification.

Chapter C requires that the direct analysis method, as described herein, be used wherever the value of the *sidesway* amplification ratio $\Delta_{2nd\ order}/\Delta_{1st\ order}$ (or B_2 from Equation C2-3), determined from a first-order analysis of the structure, exceeds 1.5. The method may also be used in lieu of the methods described in Chapter C for the analysis and design of any lateral load resisting frame in a steel building.

7.1. GENERAL REQUIREMENTS

There are potentially many parameters and behavioral effects that influence the stability of steel-framed structures (Birnstiel and Iffland, 1980; McGuire, 1992; White and Chen, 1993; ASCE Task Committee on Effective Length, 1997; Deierlein and White, 1998). Three of the most important aspects of stability behavior include geometric nonlinearities, spread-of-plasticity, and member limit states. These aspects ultimately govern frame deformations under applied loads and the resulting load effects in the structure.

Geometric Nonlinearities and Imperfections. Modern stability design provisions are based on the premise that the member forces are calculated by second-order elastic analysis, where equilibrium is satisfied on the deformed geometry of the structure. The amplification of first-order analysis forces by the traditional B_1 and B_2 factors in Chapter C is one method of conducting an approximate second-order elastic analysis. Where stability effects are significant, consideration must be given to initial geometric imperfections in the structure due to fabrication and erection tolerances. In the development and calibration of the direct analysis method, initial geometric imperfections are conservatively assumed to be equal to the maximum fabrication and erection tolerances permitted by the AISC *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005). For columns and frames, this implies a member out-of-straightness equal to $L/1000$, where L is the member length between brace or framing points, and a frame out-of-plumbness equal to $H/500$, where H is the story height. The out-of-plumbness also may be limited by the absolute bounds specified in the *Code of Standard Practice for Steel Buildings and Bridges* (AISC, 2005).

Spread of Plasticity. The direct analysis method is also calibrated against inelastic distributed-plasticity analyses that account for the spread of plasticity through the member cross-section and along the member length. The nominal thermal *residual stresses* in W-shape members are assumed to have a maximum value of $0.3F_y$ in compression at the flange tips and to be distributed according to the so-called Lehigh pattern—a linear variation across the flanges and uniform tension in the web (Deierlein and White, 1998).

Member Limit States. Member strength may be controlled by one or more of the following limit states: cross-section yielding, local buckling, flexural buckling, and lateral-torsional or *flexural-torsional buckling*. For beam-columns in single axis flexure and compression, the analysis results from the direct analysis method may be used with the new interaction equations in Chapter H, which address in-plane flexural buckling and out-of-plane lateral torsional instability separately. The separate interaction equations reduce the conservatism in the 1999 *LRFD Specification* (AISC, 2000b) provisions, which combine the two limit state checks into one equation, by using the most severe combination of in-plane or out-of-plane limits for $P_u/\phi P_n$ and $M_u/\phi M_n$. A significant advantage of the direct analysis method is that the in-plane check with P_n in the interaction equation is determined using $K = 1.0$ (in other words, $KL = L$).

Second-Order Analysis. The stability design provisions of Chapter C are developed for use with second-order elastic analysis. It is important that all component and connection deformations that contribute to the lateral displacement of the structure be considered in the analysis. In practice, there are alternative approaches one can employ for conducting second-order analyses, some of which are more rigorous than others.

Rigorous second-order analyses are those that accurately model all significant second-order effects. Rigorous analyses include solution of the governing differential equation, either through stability functions or computer frame analysis programs that model these effects (McGuire, 1992; Deierlein and White, 1998).

Many (but not all) modern commercial computer programs are capable of rigorous analyses, although this should be verified by the user for each particular program. Methods that modify first-order analysis results through second-order amplifiers (for example, B_1 and B_2 factors) are in some cases accurate enough to constitute a rigorous analysis. The use of the B_1 and B_2 amplifiers is permitted, even when $B_2 > 1.5$, provided they are determined using the reduced stiffnesses defined in Equations A-7-2 and A-7-3.

Approximate second-order analyses are those that do not meet the requirements of rigorous analysis. A common type of approximate analysis is one that captures only P-Δ effects due to member end translations (for example, *interstory drift*) but fails to capture P-δ effects due to curvature of the member relative to its chord. Where P-δ effects are significant, errors arise in approximate methods that do not accurately account for the effect of P-δ moments on amplification of both local member moments and the global (Δ) displacements. These errors can occur both with second-order computer analysis programs and with the B_1 and B_2 amplifiers. (Maleck and White, 2003) suggest an equation equivalent to Equation A-7-1 to distinguish cases where P-δ effects can be safely ignored. Alternatively, the engineer should verify the accuracy of the second-order analysis by comparisons to known solutions for conditions similar to those in the structure. Examples of the errors one may encounter are discussed in LeMessurier (1977) and Deierlein and White (1998).

It is suggested that in most building structures, the second-order *sidesway* amplification (or the equivalent B_2), calculated with the reduced stiffness, should be kept no greater than $\Delta_{2nd\ order}/\Delta_{1st\ order} = 2.5$. At larger amplification levels, small changes in gravity loads or stiffnesses result in relatively large changes in *sidesway* deflections and internal second-order forces, due to large geometric nonlinearities. Also note that stiffness requirements for control of seismic drift are included in many building codes that prohibit amplification or B_2 levels from exceeding approximately 1.5 to 1.6 (typically calculated, for steel structures, without use of a reduced stiffness) (ICC, 2003).

Effective Length Method versus the Direct Analysis Method. The effective length method for assessing member axial compressive strength, as discussed in Chapter C of this Commentary, has been used in various forms in the AISC Specification since 1961. The provisions of the current Chapter C are essentially the same as those in the 1999 *LRFD Specification* (AISC, 2000b), with the exception that: (1) limits are placed on the magnitude of second-order effects (as quantified by the ($\Delta_{2nd\ order}/\Delta_{1st\ order}$) or B_2 limit of 1.5); and (2) a minimum lateral load of $0.002Y_i$ (where Y_i is the design gravity load acting on level i) is required to be placed at each level of the structure for all gravity load-only combinations. These limits and requirements are specified for the effective length method (which uses the nominal geometry and elastic stiffness) to limit errors caused by not explicitly accounting in the analysis for initial out-of-plumbness and member stiffness reduction due to spread of plasticity. The method is based on calculating effective column buckling lengths, *KL,* which have their basis in elastic (or inelastic) stability theory. In the effective length method, the effective buckling length *KL,* or alternatively the equivalent elastic column buckling load, $P_e = \pi^2 EI/(KL)^2$, is used to calculate an axial compressive strength, P_n, through an empirical *column curve* that accounts for geometric imperfections and distributed yielding (including *residual stress* effects). This column strength is then combined with the flexural strength, M_n, and second-order member forces, P_u and M_u, in the beam-column interaction equations.

Differences between the effective length method and the direct analysis method lie predominantly in the in-plane strength check. Figure C-A-7.1(a) shows a plot of the in-plane interaction equation for the effective length method, where the anchor point on the vertical axis, P_{nKL}, is determined using an effective buckling length. Also shown in this plot is the same interaction equation with the first term based on the yield load, P_y. For W-shape members, this in-plane beam-column interaction equation is a reasonable estimate of the internal force state associated with full cross-section plastification. The P versus M response of a typical member, obtained from second-order spread-of-plasticity analysis and labeled “actual response,” indicates the maximum axial force, P_u, that the member can sustain prior to the onset of instability. The load-deflection response from a second-order elastic analysis using the nominal geometry and elastic stiffness, as conducted with the effective length method, is also shown. The “actual response” curve has larger moments than the above second-order elastic curve due to the combined effects of distributed yielding and geometric imperfections, which are not included in the second-order elastic analysis. In the effective length method, the intersection of the second-order elastic analysis curve with the P_{nKL} interaction curve determines the member strength. The plot in Figure C-A-7.1(a) shows that the effective length method is calibrated to give a resultant axial strength, P_u, consistent with the actual response. For slender columns, the calculation of the effective length *KL* (and P_{nKL}) is critical to achieving an accurate solution when using the effective length method.

While the effective length method is calibrated to accurately assess the resultant in-plane member strength, one consequence of the procedure is that it underestimates the actual internal moments under the factored loads (see Figure C-A-7.1(a)). This is inconsequential for the beam-column in-plane strength check (since P_{nKL} reduces the effective strength in the correct proportion); however, the reduced moment can affect the design of the beams and connections, which provide rotational restraint to the column. This is of greatest concern when the calculated moments are small and axial loads are large, such that P-Δ moments induced by column out-of-plumbness can be significant.

A major advantage of the direct analysis method is that it more accurately captures the internal forces in the structure, particularly for the cases where there are high gravity loads and low lateral loads. This advantage comes at the expense of applying notional lateral loads to the structure and reducing the frame stiffness as part of the analysis input.

7.2. NOTIONAL LOADS

Notional loads are lateral loads that are applied at each framing level and are specified in terms of the gravity loads applied at that level. The gravity loads used to determine the notional load must be equal to or greater than the gravity loads associated with the load combination being evaluated. Notional loads must be applied in the direction that adds to the destabilizing effects under the specified load combination.

The purpose of notional loads is to account for the destabilizing effects of geometric imperfections, nonideal conditions (such as incidental patterned gravity load effects, temperature gradients across the structure, foundation settlement, uneven column shortening, or any other effects that could induce sway that is not explicitly considered in the analysis), inelasticity in structural members, or

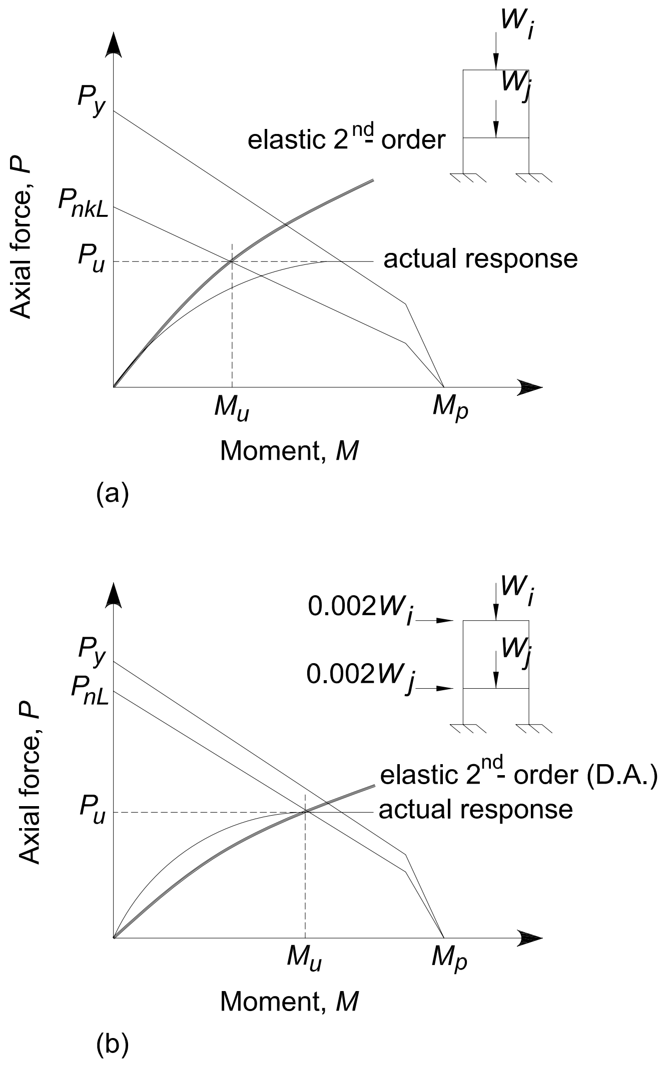


Fig. C-A-7.1. Comparison of in-plane beam-column interaction checks for (a) the effective length method and (b) the direct analysis method.

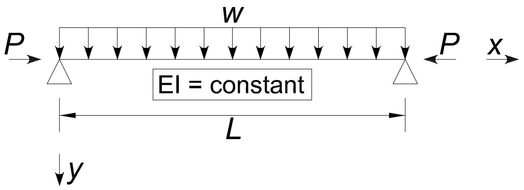
combinations thereof. While it accounts for any or all of these potential effects, the magnitude of the notional load $0.002Y_i$ can be thought of as representing an initial out-of-plumbness in each story of the structure of 1/500 times the story height. If a smaller value can be justified by the designer, it is permitted to adjust the magnitude of the notional load proportionately. Note that it is also permissible to model the structure in an assumed out-of-plumb state in lieu of applying the notional load.

7.3. DESIGN-ANALYSIS CONSTRAINTS

The direct analysis method begins with the basic requirement to calculate accurately the internal load effects using a rigorous second-order analysis. This stipulation is placed on the method to afford the luxury of using $K = 1.0$ in the first term of the beam-column interaction equation. In order to obtain accuracy in the calculation of second-order effects, certain constraints must be placed on the method as discussed below.

The first constraint (clause 1) requires that a rigorous second-order analysis be conducted that accounts for both P-Δ and P-δ effects. P-Δ effects are the effects of loads acting on the displaced location of joints or nodes in a structure. P-δ effects are the effect of loads acting on the deflected shape of a member between joints or nodes. Two benchmark problems have been established to determine whether an analysis method meets the requirements of a rigorous second-order analysis adequate for use in the direct analysis method. The problem descriptions and their rigorous differential equation solutions are shown in Figure C-A-7.2. Case 1 is a simply supported beam column subject to a uniform transverse load between supports. This problem contains only a P-δ effect since there is no translation of one end of the member relative to the other. The second problem is a flagpole column with a lateral load at its top. This problem contains both P-Δ and P-δ effects. Figure C-A-7.3 plots the results for the maximum moment and deflection as a function of the applied load P/P_{eL} using the rigorous solution. Note also that if the magnitude of the axial load on the member is less than or equal to $0.15P_{eL}$(where $P_{eL} = \pi^2 EI/L^2$), then it is permitted to ignore the P-δ effect on the lateral displacement Δ of the structure as the error in doing so is relatively small (Maleck and White, 2003). However, the P-δ effect on the internal moment in the member must be considered (see Figures C-A-7.2 and C-A-7.3). When using the benchmark problems to assess the correctness of a second-order analysis method or computer program, the computer model should utilize joints only at the ends of the member (unless joints are planned on being used along the member length in the actual structure to be modeled). Both moments and deflections should be checked at the location shown for various levels of axial load on the member (including loads that result in moment and deflection amplification, M_{max}/M_o and y_{max}/y_o, of more than 2.5) the results should agree within 3 percent. Other possible benchmark problems can be found in Chen and Lui (1987), which contains the rigorous solution for a simply-supported beam-column subject to compression and applied end moments and also a solution for a fixed-ended beam-column subject to compression and uniformly distributed loads. Typically, the calculation of accurate internal M_r values is more difficult in problems where member load and/or displacement boundary conditions are not symmetrical.

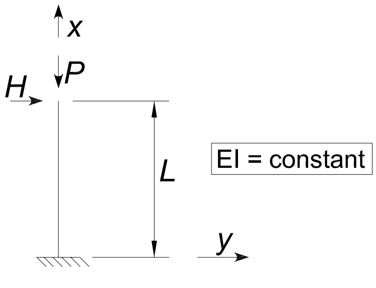
The second constraint (clause 2) requires the application of a notional load $N_i = 0.002Y_i$, where Y_i is the gravity load from the appropriate load combination acting on level i. The notional loads are required to account for the destabilizing effects of initial imperfections and other conditions that may induce sway not explicitly modeled in the structure. Note that the notional load coefficient 0.002 is based on an initial out-of-plumbness ratio from all effects equal to 1/500. Where a different value can be justified, the coefficient may be adjusted proportionately. When second-order effects are kept to a level so that the *sidesway* amplification $\Delta_{2nd\ order}/\Delta_{1st\ order}$ or $B_2 \leq 1.5$ (1.71 using the reduced elastic stiffness), then it is permitted to apply the notional loads only in the gravity load-only combinations and not in combination with other lateral loads. At this low range of *sidesway*



$$M_{MAX}\left(@x = \frac{L}{2}\right) = \frac{wL^2}{8}\left[\frac{2(\sec u - 1)}{u^2}\right] \quad \text{where } u = \sqrt{\frac{PL^2}{4EI}},\ M_o = \frac{wL^2}{8}$$

$$y_{MAX}\left(@x = \frac{L}{2}\right) = \frac{5wL^4}{384EI}\left[\frac{12(2\sec u - u^2 - 2)}{5u^4}\right] \quad \text{where } y_o = \frac{5wL^4}{384EI}$$

Case 1



$$M_{MAX}\,(@x = 0) = HL\left(\frac{\tan\alpha}{\alpha}\right) \quad \text{where } \alpha = \sqrt{\frac{PL^2}{EI}},\ \ M_o = HL$$

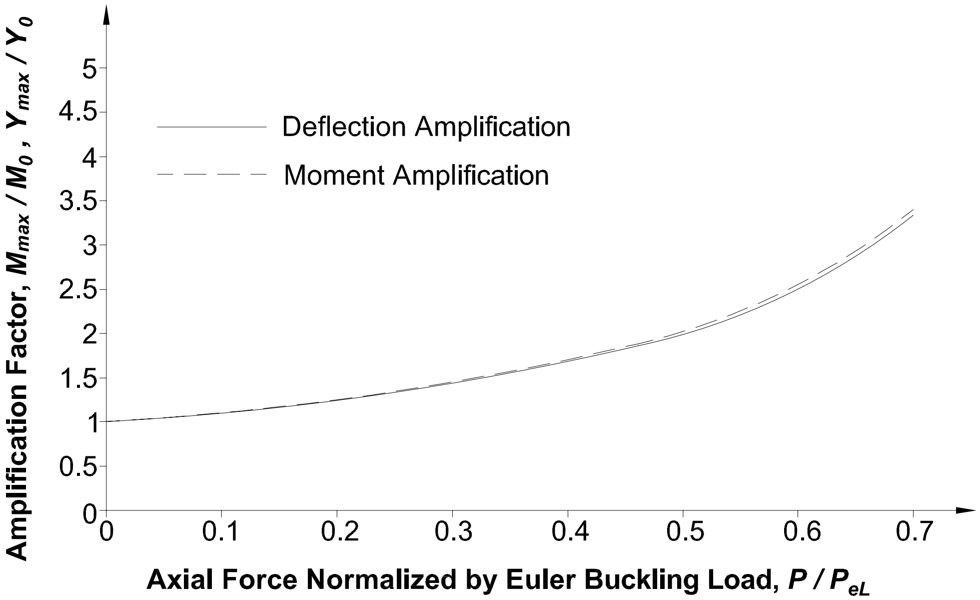
$$y_{MAX}\,(@x = L) = \frac{HL^3}{3EI}\left(\frac{3(\tan\alpha - \alpha)}{\alpha^3}\right) \quad \text{where } y_o = \frac{HL^3}{3EI}$$

Case 2

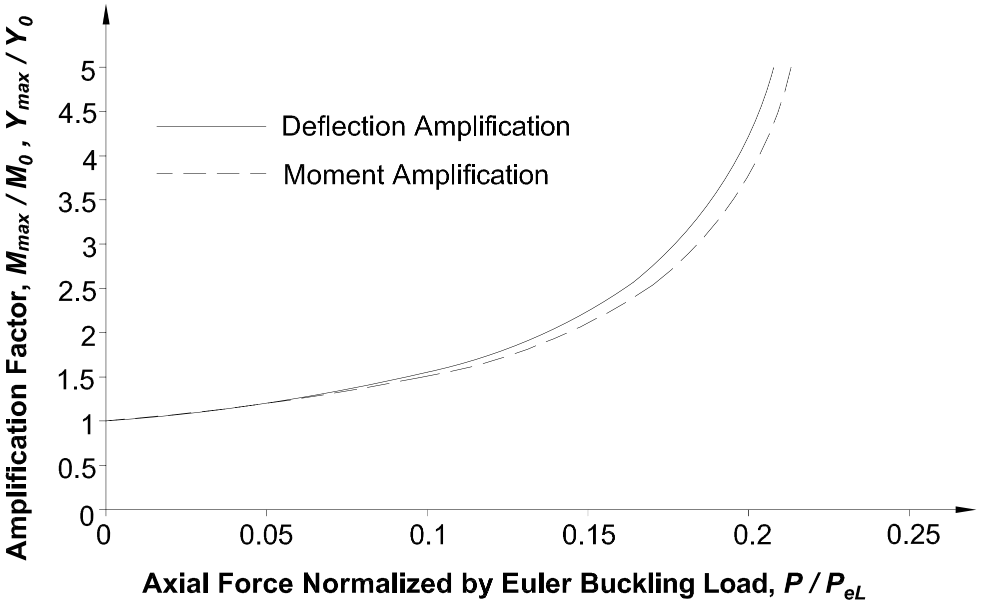
Fig. C-A-7.2. Benchmark problems.

amplification or B_2, the resulting errors in the internal forces are relatively small. If the notional loads are applied in combination with other lateral loads, there is no need for checking a B_2 limit. In all cases it is permitted to use the assumed out-of-plumbness geometry in the analysis of the structure in lieu of applying notional loads as an acceptable way to account for the geometric imperfection effects.

The third constraint (clauses 3 and 4) requires that the analysis be based on a reduced stiffness ($EI^* = 0.8\tau_b EI$ and $EA^* = 0.8EA$) in the structure. There are two reasons for imposing the reduced stiffness for analysis. First, for frames with slender members, where the limit state is governed by elastic stability, the



Case 1



Case 2

Fig. C-A-7.3. Maximum moment and deflection values as a function of axial force for benchmark problems.

0.8 factor on stiffness results in a system available strength equal to 0.8 times the elastic stability limit. This is roughly equivalent to the margin of safety implied by design of slender columns by the effective length procedure where the design strength $\phi P_n = 0.9(0.877)P_e = 0.79P_e$ where P_e is the elastic *critical load*, 0.90 is the specified resistance factor, and 0.877 is a reduction factor in the *column curve* equation (Equation E3-3). Second, for frames with intermediate or stocky columns, the $0.8\tau_b$ factor reduces the stiffness to account for inelastic softening prior to the members reaching their design strength. The τ_b factor is similar to the inelastic stiffness reduction factor implied in the *column curve* to account for loss of stiffness under high compression loads ($P_u > 0.5P_y$),and the 0.8 factor accounts for additional softening under combined axial compression and bending. It is a fortuitous coincidence that the reduction coefficients for both slender and stocky columns are close enough, such that the single reduction factor of $0.8\tau_b$works over the full range of slenderness. The reduced stiffness and notional load requirements only pertain to analyses for strength limit states. They do not apply to analyses of serviceability conditions of excessive deflections, vibration, etc. For ease of application in design practice, where $\tau_b = 1$, the reduction on EI and EA can be applied by modifying E in the analysis. However, for computer programs that do semi-automated design, one should ascertain that the reduced E is applied only for the second-order analysis. The elastic modulus should not be reduced in nominal strength equations that include E (for example, M_n for laterally unbraced beams). As shown in Figure C-A-7.1(b), the net effect of modifying the analysis in the manner just described is to amplify the second-order forces such that they are closer to the actual internal forces in the structure. It is for this reason that the beam-column interaction for in-plane flexural buckling is checked using an axial strength P_{nL} calculated from the *column curve* using the actual unbraced member length L, in other words, with $K = 1.0$.

In cases where the flexibility of other structural components (for example, connections, flexible column base details, or horizontal trusses acting as diaphragms) is modeled explicitly in the analysis, the stiffness of the other structural components should be reduced as well. Conservatively, the stiffness reduction may be taken as $EA^* = 0.8EA$ and/or $EI^* = 0.8EI$ for all cases. Surovek-Maleck, White, and Leon (2004) discuss the appropriate reduction of connection stiffnesses in the analysis of PR frames.

Simplified First-Order Analysis Based on the Direct Analysis Method (K = 1.0). The direct analysis method provides the technical basis for the provisions of Section C2.2b for design by first-order elastic analysis with $K = 1.0$ (Kuchenbecker, White, and Surovek-Maleck, 2004). The method is based on an assumed out-of-plumbness in the structure $\Delta_o/L = 0.002$, a target maximum drift ratio Δ/L, and reduced stiffnesses in the frame members ($0.8\tau_b EI$ and $0.8EA$). The first-order analysis is carried out using the nominal (unreduced) stiffness, and the above stiffness reduction is accounted for solely within the calculation of amplification factors. The method is applicable to braced, moment and combined frames. This method has a number of distinct advantages compared to the amplified first-order elastic approach specified in Chapter C:

(1) The second-order internal forces and moments are determined directly as part of the first-order analysis.
(2) There is no need to subdivide the analysis into artificial NT and LT parts.

Kuchenbecker and others (2004) present a general form of the suggested method. If the above approach is employed, it can be shown that for $B_2 \leq 1.5$ and $\tau_b = 1.0$ the required additional lateral load to be applied with other lateral loads in a first-order analysis of the structure, using the nominal (unreduced) stiffness, can be determined as:

$$N_i = \left(\frac{B_2}{1 - 0.2B_2}\right)\frac{\Delta}{\mathrm{L}}Y_i \geq \left(\frac{B_2}{1 - 0.2B_2}\right) 0.002Y_i \qquad \text{(C-A-7-3-1)}$$

where B_2 and Y_i are as defined in Chapter C, and Δ_H/L is the target maximum first-order drift ratio due to either the LRFD strength load combinations or 1.6 times the ASD strength load combinations. Note that if B_2 (based on the unreduced stiffness) is set to the 1.5 limit prescribed in Chapter C, then,

$$N_i = 2.1\,(\Delta/L)Y_i \geq 0.0042\ Y_i \qquad \text{(C-A-7-3-2)}$$

This is the additional lateral load required in Section C2.2b(2) of Chapter C.

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Specification for Structural Joints Using ASTM A325 or A490 Bolts

June 30, 2004

Supersedes the June 23, 2000 *Specification for Structural Joints Using ASTM A325 or A490 Bolts.*

Prepared by RCSC Committee A.1—Specifications and approved by the Research Council on Structural Connections.

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PREFACE

The purpose of the Research Council on Structural Connections (RCSC) is:

(1) To stimulate and support such investigation as may be deemed necessary and valuable to determine the suitability, strength and behavior of various types of structural connections;
(2) To promote the knowledge of economical and efficient practices relating to such structural connections; and,
(3) To prepare and publish related standards and such other documents as necessary to achieving its purpose.

The Council membership consists of qualified structural engineers from academic and research institutions, practicing design engineers, suppliers and manufacturers of fastener components, fabricators, erectors and code-writing authorities.

The first Specification approved by the Council, called the *Specification for Assembly of Structural Joints Using High Tensile Steel Bolts*, was published in January 1951. Since that time the Council has published fifteen successive editions. Each was developed through the deliberations and approval of the full Council membership and based upon past successful usage, advances in the state of knowledge and changes in engineering design practice. This edition of the Council's *Specification for Structural Joints Using ASTM A325 or A490 Bolts* continues the tradition of earlier editions. The major changes are:

- Sections 5.1, 5.2, and 5.3 were editorially revised to clarify strength requirements of slip critical connections.
- Section 6.2.1 was modified to permit the use of A490 type bolts, with round heads equal or larger in diameter than ASTM F1852 heads, without F436 hardened washers.
- Table 6.1, footnote d, was added to clarify use of non-hardened plate washer to be used in conjunction with an ASTM F436 hardened washer.
- Commentary Table C-2.1 bolt head and nut dimension locations F and W as shown in the artwork Figure C-2.2 was corrected.

In addition, typographical changes have been made throughout this Specification.

By the Research Council on Structural Connections,

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SYMBOLS

The following symbols are used in this Specification.

A_b	Cross-sectional area based upon the nominal diameter of bolt, in.^{2}
D	Slip probability factor as described in Section 5.4.2
D_u	Multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension T_m as described in Section 5.4.1
F_n	Nominal strength (per unit area), ksi
F_u	Specified minimum tensile strength (per unit area), ksi
I	Moment of inertia of the built-up member about the axis of buckling (see the Commentary to Section 5.4), in.^{4}
L	Total length of the built-up member (see the Commentary to Section 5.4), in.
L_c	Clear distance, in the direction of load, between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.
N_b	Number of bolts in the joint
P_u	Required strength in compression, kips; Axial compressive force in the built-up member (see the Commentary to Section 5.4), kips
Q	First moment of area of one component about the axis of buckling of the built-up member (see the Commentary to Section 5.4), in.^{3}
R_n	Nominal strength, kips
R_s	Service-load slip resistance, kips
T	Applied service load in tension, kips
T_m	Specified minimum bolt pretension (for pretensioned joints as specified in Table 8.1), kips
T_u	Required strength in tension (factored tensile load), kips
V_u	Required strength in shear (factored shear load), kips
d_b	Nominal diameter of bolt, in.
t	Thickness of the connected material, in.

t' Total thickness of fillers or shims (see Section 5.1), in.

k_s Slip coefficient for an individual specimen determined in accordance with Appendix A

ϕ Resistance factor

ϕR_n Design strength, kips

μ Mean slip coefficient

GLOSSARY

The following terms are used in this Specification. Where used, they are italicized to alert the user that the term is defined in this Glossary.

Coated Faying Surface. A *faying surface* that has been primed, primed and painted or protected against corrosion, except by hot-dip galvanizing.

Connection. An assembly of one or more *joints* that is used to transmit forces between two or more members.

Contractor. The party or parties responsible to provide, prepare and assemble the fastener components and connected parts described in this Specification.

Design Strength. ϕR_n, the resistance provided by an element or *connection*; the product of the *nominal strength* R_n and the resistance factor ϕ.

Engineer of Record. The party responsible for the design of the structure and for the approvals that are required in this Specification (see Section 1.4 and the corresponding Commentary).

Fastener Assembly. An assembly of fastener components that is supplied, tested and installed as a unit.

Faying Surface. The plane of contact between two plies of a *joint*.

Firm Contact. The condition that exists on a *faying surface* when the plies are solidly seated against each other, but not necessarily in continuous contact.

Galvanized Faying Surface. A *faying surface* that has been hot-dip galvanized.

Grip. The total thickness of the plies of a *joint* through which the bolt passes, exclusive of washers or direct-tension indicators.

Guide. The *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd Edition (Kulak et al., 1987).

High-Strength Bolt. An ASTM A325 or A490 bolt, an ASTM F1852 twist-off-type tension-control bolt or an alternative-design fastener that meets the requirements in Section 2.8.

Inspector. The party responsible to ensure that the *contractor* has satisfied the provisions of this Specification in the work.

Joint. A bolted assembly with or without collateral materials that is used to join two structural elements.

Lot. In this Specification, the term *lot* shall be taken as that given in the ASTM Standard as follows:

Product	ASTM Standard	See Lot Definition in Section
Bolts	A325	9.4
	A490	11.3.2 or 11.4.2
Twist-off-type tension control bolt assemblies	F1852	13.4
Nuts	A563	9.2
Washers	F436	9.2
Compressible-washer-type direct tension indicators	F959	10.2.2

Manufacturer. The party or parties that produce the components of the *fastener assembly*.

Mean Slip Coefficient. μ, the ratio of the frictional shear load at the *faying surface* to the total normal force when slip occurs.

Nominal Strength. The capacity of a structure or component to resist the effects of loads, as determined by computations using the specified material strengths and dimensions and equations derived from accepted principles of structural mechanics or by field tests or laboratory tests of scaled models, allowing for modeling effects and differences between laboratory and field conditions.

Pretensioned Joint. A *joint* that transmits shear and/or tensile loads in which the bolts have been installed in accordance with Section 8.2 to provide a pretension in the installed bolt.

Protected Storage. The continuous protection of fastener components in closed containers in a protected shelter as described in the Commentary to Section 2.2.

Prying Action. Lever action that exists in *connections* in which the line of application of the applied load is eccentric to the axis of the bolt, causing deformation of the fitting and an amplification of the axial tension in the bolt.

Required Strength. The load effect acting on an element or *connection* determined by structural analysis from the factored loads using the most appropriate critical load combination.

Routine Observation. Periodic monitoring of the work in progress.

Shear/Bearing Joint. A *snug-tightened joint* or *pretensioned joint* with bolts that transmit shear loads and for which the design criteria are based upon the shear strength of the bolts and the bearing strength of the connected materials.

Slip-Critical Joint. A *joint* that transmits shear loads or shear loads in combination with tensile loads in which the bolts have been installed in accordance with Section 8.2 to provide a pretension in the installed bolt (clamping force on the *faying surfaces*), and with *faying surfaces* that have been prepared to provide a calculable resistance against slip.

Snug-Tightened Joint. A *joint* in which the bolts have been installed in accordance with Section 8.1. The snug-tightened condition is the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the plies into *firm contact*.

Start of Work. Any time prior to the installation of *high-strength bolts* in structural *connections* in accordance with Section 8.

Sufficient Thread Engagement. Having the end of the bolt extending beyond or at least flush with the outer face of the nut; a condition that develops the strength of the bolt.

Supplier. The party that sells the fastener components to the party that will install them in the work.

Tension Calibrator. A calibrated tension-indicating device that is used to verify the acceptability of the pretensioning method when a *pretensioned joint* or *slip-critical joint* is specified.

Uncoated Faying Surface. A *faying surface* that has neither been primed, painted, nor galvanized and is free of loose scale, dirt and other foreign material.

NOTES

SPECIFICATION FOR STRUCTURAL JOINTS USING ASTM A325 OR A490 BOLTS

June 30, 2004

SECTION 1. GENERAL REQUIREMENTS

1.1. Scope

This Specification covers the design of bolted *joints* and the installation and inspection of the assemblies of fastener components listed in Section 1.3, the use of alternative-design fasteners as permitted in Section 2.8 and alternative washer-type indicating devices as permitted in Section 2.6.2, in structural steel *joints*. This Specification relates only to those aspects of the connected materials that bear upon the performance of the fastener components. The Symbols, Glossary and Appendices are a part of this Specification.

Commentary:

This Specification deals principally with two strength grades of *high-strength bolts*, ASTM A325 and A490, and with their design, installation and inspection in structural steel *joints*. Equivalent fasteners, however, such as ASTM F1852 (equivalent to ASTM A325) twist-off-type tension-control bolt assemblies, are also covered. These provisions may not be relied upon for high-strength fasteners of other chemical composition, mechanical properties, or size. These provisions do not apply when material other than steel is included in the *grip*; nor are they applicable to anchor rods.

This Specification relates only to the performance of fasteners in structural steel *joints* and those few aspects of the connected material that affect this performance. Many other aspects of *connection* design and fabrication are of equal importance and must not be overlooked. For more general information on design and issues relating to *high-strength bolting* and the connected material, refer to current steel design textbooks and the *Guide to Design Criteria for Bolted and Riveted Joints*, 2nd Edition (Kulak et al., 1987).

1.2. Loads, Load Factors and Load Combinations

The design and construction of the structure shall conform to an applicable load and resistance factor design specification for steel structures. Because factored load combinations account for the reduced probabilities of maximum loads acting concurrently, the *design strengths* given in this Specification shall not be increased. Appendix B is included as an alternative approach.

Commentary:

This Specification is written in the load and resistance factor design (LRFD) format, which provides a method of proportioning structural components such that no applicable limit state is exceeded when the structure is subject to all appropriate load combinations. When a structure or structural component ceases to fulfill the intended purpose in some way, it is said to have exceeded a limit state. Strength limit states concern maximum load-carrying capability, and are related to safety. Serviceability limit states are usually related to performance under normal service conditions, and usually are not related to strength or safety. The term “resistance” includes both strength limit states and serviceability limit states.

The *design strength* ϕR_n is the *nominal strength* R_n multiplied by the resistance factor ϕ. The factored load is the sum of the nominal loads multiplied by load factors, with due recognition of load combinations that account for the improbability of simultaneous occurrence of multiple transient load effects at their respective maximum values. The *design strength* ϕR_n of each structural component or assemblage must equal or exceed the *required strength* (V_u, T_u, etc.).

Although loads, load factors and load combinations are not explicitly specified in this Specification, the resistance factors herein are based upon those specified in ASCE 7. When the design is governed by other load criteria, the resistance factors specified herein should be adjusted as appropriate.

1.3. Referenced Standards and Specifications

The following standards and specifications are referenced herein:

American Institute of Steel Construction
Load and Resistance Factor Design Specification for Structural Steel Buildings, December 27,1999

American National Standards Institute
ANSI/ASME B18.2.6-96 *Fasteners for Use in Structural Applications*

American Society for Testing and Materials
ASTM A123-97a *Standard Specification for Zinc (Hot-Dip Galvanized) Coatings on Iron and Steel Products*

ASTM A153-98 *Standard Specification for Zinc Coating (Hot-Dip) on Iron and Steel Hardware*

ASTM A194-98b *Specification for Carbon and Alloy Steel Nuts for Bolts for High Pressure or High-Temperature Service, or Both*

ASTM A325-97 *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

ASTM A490-97 *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*

ASTM A563-97 *Standard Specification for Carbon and Alloy Steel Nuts*

ASTM B695-91[1] *Standard Specification for Coatings of Zinc Mechanically Deposited on Iron and Steel*

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[1] Reapproved 1997.

ASTM F436-93 *Standard Specification for Hardened Steel Washers*

ASTM F959-99a *Standard Specification for Compressible-Washer-Type Direct Tension Indicators for Use with Structural Fasteners*

ASTM F1852-98 "*Twist off*" *Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*

American Society of Civil Engineers
ASCE 7-98 *Minimum Design Loads for Buildings and Other Structures*

SSPC: The Society for Protective Coatings
SSPC-PA2-96 *Measurement of Dry Coating Thickness With Magnetic Gages*

Commentary:
Familiarity with the referenced AISC, ASCE, ASME, ASTM and SSPC specification requirements is necessary for the proper application of this Specification. The discussion of referenced specifications in this Commentary is limited to only a few frequently overlooked or misunderstood items.

1.4. Drawing Information

The *Engineer of Record* shall specify the following information in the contract documents

(1) The ASTM designation and type (Section 2) of bolt to be used;
(2) The *joint* type (Section 4);
(3) The required class of slip resistance if *slip-critical joints* are specified (Section 4); and,
(4) Whether slip is checked at the factored-load level or the service-load level, if *slip-critical joints* are specified (Section 5).

Commentary:
A summary of the information that the *Engineer of Record* is required to provide in the contract documents is provided in this Section. The parenthetical reference after each listed item indicates the location of the actual requirement in this Specification. In addition, the approval of the *Engineer of Record* is required in this Specification in the following cases:

(1) For the reuse of non-galvanized ASTM A325 bolts (Section 2.3.3);
(2) For the use of alternative washer-type indicating devices that differ from those that meet the requirements of ASTM F959, including the corresponding installation and inspection requirements that are provided by the *manufacturer* (Section 2.6.2);
(3) For the use of alternative-design fasteners, including the corresponding installation and inspection requirements that are provided by the *manufacturer* (Section 2.8);

(4) For the use of faying-surface coatings in *slip-critical joints* that provide a *mean slip coefficient* determined per Appendix A, but differing from Class A or Class B (Section 3.2.2(b));

(5) For the use of thermal cutting in the production of bolt holes (Section 3.3);

(6) For the use of oversized (Section 3.3.2), short-slotted (Section 3.3.3) or long slotted holes (Section 3.3.4) in lieu of standard holes;

(7) For the use of a value of D_u other than 1.13 (Section 5.4.1); and,

(8) For the use of a value of D other than 0.80 (Section 5.4.2).

SECTION 2. FASTENER COMPONENTS

2.1. Manufacturer Certification of Fastener Components

Manufacturer certifications documenting conformance to the applicable specifications required in Sections 2.3 through 2.8 for all fastener components used in the *fastener assemblies* shall be available to the *Engineer of Record* and *inspector* prior to assembly or erection of structural steel.

Commentary:
Certification by the *manufacturer* or *supplier* of *high-strength bolts*, nuts, washers and other components of the *fastener assembly* is required to ensure that the components to be used are identifiable and meet the requirements of the applicable ASTM Specifications.

2.2. Storage of Fastener Components

Fastener components shall be protected from dirt and moisture in closed containers at the site of installation. Only as many fastener components as are anticipated to be installed during the work shift shall be taken from *protected storage*. Fastener components that are not incorporated into the work shall be returned to *protected storage* at the end of the work shift. Fastener components shall not be cleaned or modified from the as-delivered condition.

Fastener components that accumulate rust or dirt shall not be incorporated into the work unless they are requalified as specified in Section 7. ASTM F1852 twist-off-type tension-control bolt assemblies and alternative-design fasteners that meet the requirements in Section 2.8 shall not be relubricated, except by the *manufacturer*.

Commentary:
Protected storage requirements are specified for *high-strength bolts*, nuts, washers and other fastener components with the intent that the condition of the components be maintained as nearly as possible to the as-manufactured condition until they are installed in the work. This involves:

(1) The storage of the fastener components in closed containers to protect from dirt and corrosion;
(2) The storage of the closed containers in a protected shelter;
(3) The removal of fastener components from *protected storage* only as necessary; and,
(4) The prompt return of unused fastener components to *protected storage*.

To facilitate manufacture, prevent corrosion and facilitate installation, the *manufacturer* may apply various coatings and oils that are present in the as manufactured condition. As such, the condition of supplied fastener components or the *fastener assembly* should not be altered to make them unsuitable for pre-tensioned installation.

If fastener components become dirty, rusty, or otherwise have their as received condition altered, they may be unsuitable for pre-tensioned installation.

It is also possible that a *fastener assembly* may not pass the pre-installation verification requirements of Section 7. Except for ASTM F1852 twist-off-type tension-control bolt assemblies (Section 2.7) and some alternative-design fasteners (Section 2.8), fastener components can be cleaned and lubricated by the fabricator or the erector. Because the acceptability of their installation is dependent upon specific lubrication, ASTM F1852 twist-off-type tension-control bolt assemblies and some alternative-design fasteners are suitable only if the *manufacturer* lubricates them.

2.3. Heavy-Hex Structural Bolts

2.3.1. Specifications: Heavy-hex structural bolts shall meet the requirements of ASTM A325 or ASTM A490. The *Engineer of Record* shall specify the ASTM designation and type of bolt (see Table 2.1) to be used.

2.3.2. Geometry: Heavy-hex structural bolt dimensions shall meet the requirements of ANSI/ASME B18.2.6. The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed.

Table 2.1. Acceptable ASTM A563 Nut Grade and Finish and ASTM F436 Washer Type and Finish

ASTM Desig.	Bolt Type	Bolt Finish[d]	ASTM A563 nut grade and finish[d]	ASTM F436 washer type and finish[a,d]
A325	**1**	**Plain (uncoated)**	C, C3, D, DH[c] and DH3; plain	1; plain
		Galvanized	DH[c]; galvanized And lubricated	1; galvanized
	3	**Plain**	C3 and DH3; plain	3; plain
F1852	**1**	**Plain (uncoated)**	C, C3, DH[c] and DH3; plain	1; plain[b]
		Mechanically Galvanized	DH[c]; mechanically galvanized and lubricated	1; mechanically galvanized[b]
	3	**Plain**	C3 and DH3; plain	3; plain[b]
A490	**1**	**Plain**	DH[c] and DH3; plain	1; plain
	3	**Plain**	DH3; plain	3; plain

[a] Applicable only if washer is required in Section 6.

[b] Required in all cases under nut per Section 6.

[c] The substitution of ASTM A194 grade 2H nuts in place of ASTM A563 grade DH nuts is permitted.

[d] "Galvanized" as used in this table refers to hot-dip galvanizing in accordance with ASTM A153 or mechanical galvanizing in accordance with ASTM B695.

2.3.3. Reuse: ASTM A490 bolts and galvanized ASTM A325 bolts shall not be reused. When approved by the *Engineer of Record*, black ASTM A325 bolts are permitted to be reused. Touching up or re-tightening bolts that may have been loosened by the installation of adjacent bolts shall not be considered to be a reuse.

Commentary:
ASTM A325 and ASTM A490 currently provide for two types (according to metallurgical classification) of *high-strength bolts*, supplied in diameters from $^1/_2$ in. to $1^1/_2$ in. inclusive. Type 1 covers medium carbon steel for ASTM A325 bolts and alloy steel for ASTM A490 bolts. Type 3 covers *high-strength bolts* that have improved atmospheric corrosion resistance and weathering characteristics. (Reference to Type 2 ASTM A325 and Type 2 A490 bolts, which appeared in previous editions of this Specification, has been removed following the removal of similar reference within the ASTM A325 and A490 Specifications). When the bolt type is not specified, either Type 1 or Type 3 may be supplied at the option of the *manufacturer*. Note that ASTM F1852 twist-off-type tension-control bolt assemblies may be manufactured with a button head or hexagonal head; other requirements for these *fastener assemblies* are found in Section 2.7.

Regular heavy-hex structural bolts and twist-off-type tension-control bolt assemblies are required by ASTM Specifications to be distinctively marked. Certain markings are mandatory. In addition to the mandatory markings, the *manufacturer* may apply additional distinguishing markings. The mandatory and sample optional markings are illustrated in Figure C-2.1.

ASTM Specifications permit the galvanizing of ASTM A325 bolts but not ASTM A490 bolts. Similarly, the application of zinc to ASTM A490 bolts by metallizing or mechanical coating is not permitted because the effect of mechanical galvanizing on embrittlement and delayed cracking of ASTM A490 bolts has not been fully investigated to date.

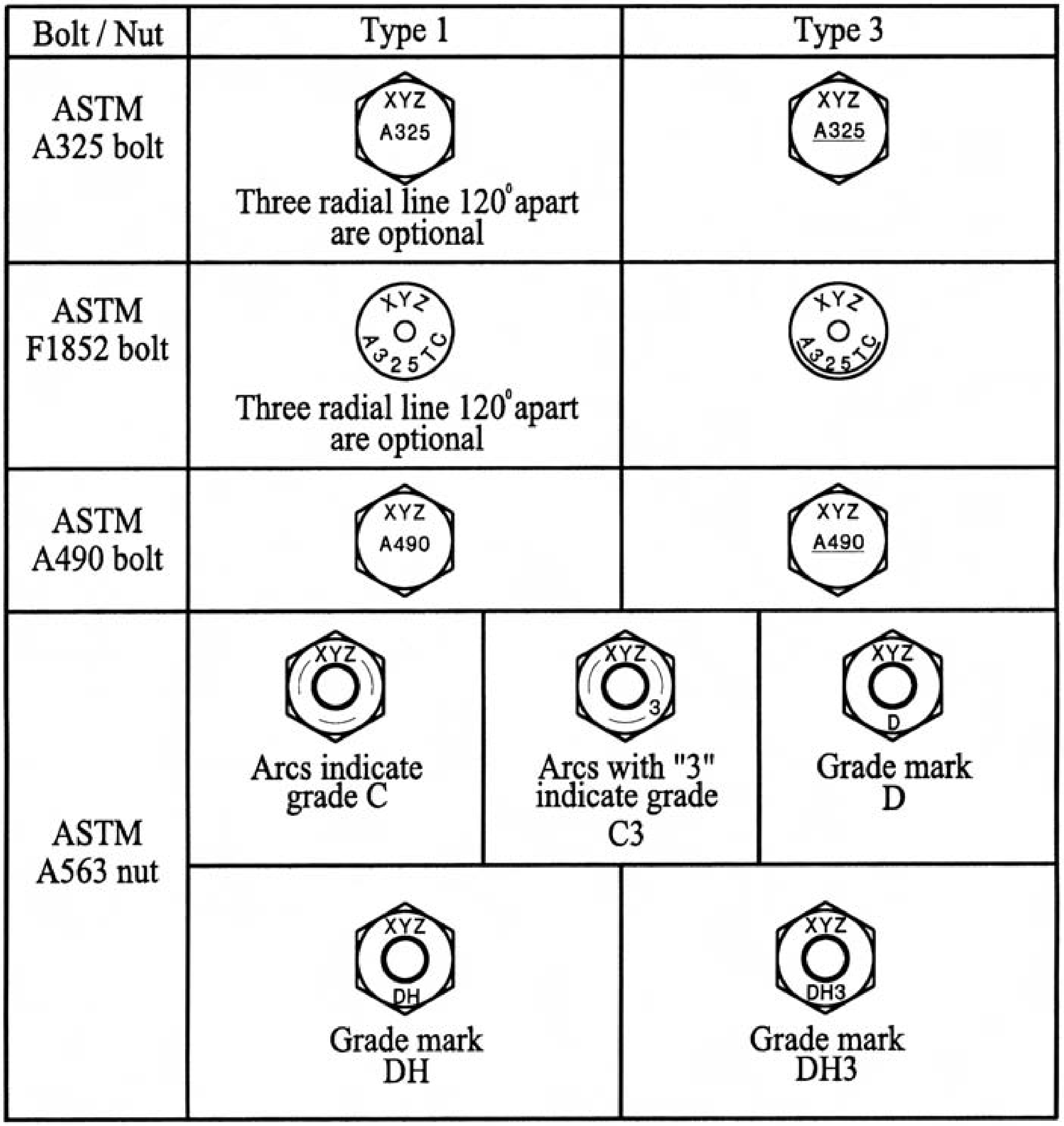
Galvanized *high-strength bolts* and nuts must be considered as a manufactured *fastener assembly*. Insofar as the hot-dip galvanized bolt and nut assembly is concerned, four principal factors must be considered so that the provisions of this Specification are understood and properly applied. These are:

(1) The effect of the hot-dip galvanizing process on the mechanical properties of high-strength steels;
(2) The effect of over-tapping for hot-dip galvanized coatings on the nut stripping strength;
(3) The effect of galvanizing and lubrication on the torque required for pretensioning; and,
(4) Shipping requirements.

Birkemoe and Herrschaft (1970) showed that, in the as-galvanized condition, galvanizing increases the friction between the bolt and nut threads as well as the variability of the torque-induced pretension. A lower required torque and more consistent results are obtained if the nuts are lubricated. Thus, it is required in ASTM A325 that a galvanized bolt and lubricated galvanized nut be assembled in a steel *joint* with a galvanized washer and tested by the *supplier* prior to shipment. This testing must show that the galvanized nut with the lubricant provided may be rotated from the snug-tight condition well in excess of the rotation required for pretensioned installation without stripping. This requirement applies to both hot-dip and mechanically galvanized fasteners. The above requirements clearly indicate that:

(1) Galvanized *high-strength bolts* and nuts must be treated as a *fastener assembly*;

(2) The *supplier* must supply nuts that have been lubricated and tested with the supplied *high-strength bolts*;



Notes:
1. XYZ represents the manufacturer's identification mark.
2. ASTM F1852 twist-off-type tension-control bolt assemblies are also produced with heavy-hex head that has similar markings.

Figure C-2.1. Required marks for acceptable bolt and nut assemblies.

(3) Nuts and *high-strength bolts* must be shipped together in the same shipping container; and,

(4) The purchase of galvanized *high-strength bolts* and galvanized nuts from separate *suppliers* is not in accordance with the intent of the ASTM Specifications because the control of over-tapping, the testing and application of lubricant and the *supplier* responsibility for the performance of the assembly would clearly not have been provided as required.

Because some of the lubricants used to meet the requirements of ASTM Specifications are water soluble, it is advisable that galvanized *high-strength bolts* and nuts be shipped and stored in plastic bags or in sealed wood or metal containers. Containers of fasteners with hot-wax-type lubricants should not be subjected to heat that would cause depletion or change in the properties of the lubricant.

Both the hot-dip galvanizing process (ASTM A153) and the mechanical galvanizing process (ASTM B695) are recognized in ASTM A325. The effects of the two processes upon the performance characteristics and requirements for proper installation are distinctly different. Therefore, distinction between the two must be noted in the comments that follow. In accordance with ASTM A325, all threaded components of the *fastener assembly* must be galvanized by the same process and the *supplier's* option is limited to one process per item with no mixed processes in a *lot*. Mixing *high-strength bolts* that are galvanized by one process with nuts that are galvanized by the other may result in an unworkable assembly.

Steels in the 200 ksi and higher tensile-strength range are subject to embrittlement if hydrogen is permitted to remain in the steel and the steel is subjected to high tensile stress. The minimum tensile strength of ASTM A325 bolts is 105 ksi or 120 ksi, depending upon the diameter, and maximum hardness limits result in production tensile strengths well below the critical range. The maximum tensile strength for ASTM A490 bolts was set at 170 ksi to provide a little more than a ten-percent margin below 200 ksi. However, because *manufacturers* must target their production slightly higher than the required minimum, ASTM A490 bolts close to the critical range of tensile strength must be anticipated. For black *high-strength bolts*, this is not a cause for concern. However, if the bolt is hot-dip galvanized, delayed brittle fracture in service is a concern because of the possibility of the introduction of hydrogen during the pickling operation of the hot-dip galvanizing process and the subsequent "sealing-in" of the hydrogen by the zinc coating. There also exists the possibility of cathodic hydrogen absorption arising from the corrosion process in certain aggressive environments.

ASTM A325 and A490 bolts are manufactured to dimensions as specified in ANSI/ASME B18.2.6. The basic dimensions, as defined in Figure C-2.2, are shown in Table C-2.1.

The principal geometric features of heavy-hex structural bolts that distinguish them from bolts for general application are the size of the head and the unthreaded body length. The head of the heavy-hex structural bolt is specified to be the same size as a heavy-hex nut of the same nominal diameter so that the ironworker may use the same wrench or socket either on the bolt head and/or on the nut. With the specific exception of fully threaded ASTM A325T bolts as discussed below, heavy-hex structural bolts have shorter threaded lengths than bolts for general applications. By making the body length of the bolt the control dimension, it has been possible to exclude the thread from all shear planes when desirable, except for the case of thin outside parts adjacent to the nut.

The shorter threaded lengths provided with heavy-hex structural bolts tend to minimize the threaded portion of the bolt within the *grip*. Accordingly, care must also be exercised to provide adequate threaded length between the nut and the bolt head to enable appropriate installation without jamming the nut on the thread run-out.

Depending upon the increments of supplied bolt lengths, the full thread may extend into the *grip* for an assembly without washers by as much as 3/8 in. for 1/2, 5/8, 3/4, 7/8, 1 1/4, and 1 1/2 in. diameter *high-strength bolts* and as much as 1/2 in. for 1,

Table C-2.1. Bolt and Nut Dimensions

Nominal Bolt Diameter d_b, in.	Heavy Hex Structural Bolt Dimensions			Heavy Hex Nut Dimensions	
	Width across flats *F*, in.	Height H_1, in.	Thread Length *T*, in.	Width across flats *W*, in.	Height H_2, in.
1/2	7/8	5/16	1	7/8	31/64
5/8	1 1/16	25/64	1 1/4	1 1/16	39/64
3/4	1 1/4	15/32	1 3/8	1 1/4	47/64
7/8	1 7/16	35/64	1 1/2	1 7/16	55/64
1	1 5/8	39/64	1 3/4	1 5/8	63/64
1 1/8	1 13/16	11/16	2	1 13/16	1 7/64
1 1/4	2	25/32	2	2	1 7/32
1 3/8	2 3/16	27/32	2 1/4	2 3/16	1 11/32
1 1/2	2 3/8	15/16	2 1/4	2 3/8	1 15/32

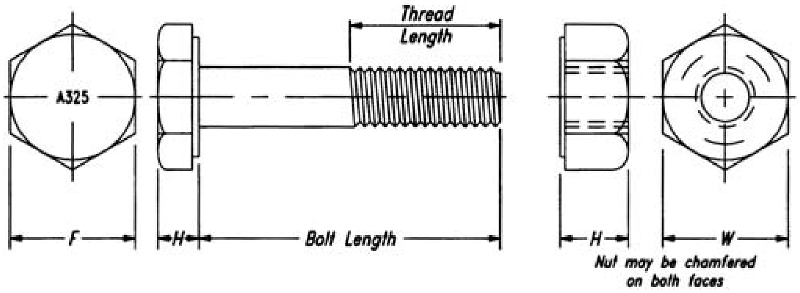


Figure C-2.2. Heavy-hex structural bolt and heavy-hex nut.

1⅛, and 1⅜ in. diameter *high-strength bolts*. When the thickness of the ply closest to the nut is less than the ⅜ in. or ½ in. dimensions given above, it may still be possible to exclude the threads from the shear plane, when required, depending upon the specific combination of bolt length, *grip* and number of washers used under the nut (Carter, 1996). If necessary, the next increment of bolt length can be specified with ASTM F436 washers in sufficient number to both exclude the threads from the shear plane and ensure that the assembly can be installed with adequate threads included in the *grip* for proper installation.

At maximum accumulation of tolerances from all components in the *fastener assembly*, the thread run-out will cross the shear plane for the critical combination of bolt length and *grip* used to select the foregoing rules of thumb for ply thickness required to exclude the threads. This condition is not of concern, however, for two reasons. First, it is too unlikely that all component tolerances will accumulate at their maximum values to warrant consideration. Second, even if the maximum accumulation were to occur, the small reduction in shear strength due to the presence of the thread run-out (not a full thread) would be negligible.

There is an exception to the foregoing thread length requirements for ASTM A325 bolts but not for ASTM A490 bolts nor ASTM F1852 twist-off-type tension-control bolt assemblies. Supplementary requirements in ASTM A325 permit the purchaser to specify a bolt that is threaded for the full length of the shank, when the bolt length is equal to or less than four times the nominal diameter. This exception is provided to increase economy through simplified ordering and inventory control in the fabrication and erection of some structures. It is particularly useful in those structures in which the strength of the *connection* is dependent upon the bearing strength of relatively thin connected material rather than the shear strength of the bolt, whether with threads in the shear plane or not. As required in ASTM A325, *high-strength bolts* ordered to such supplementary requirements must be marked with the symbol A325T.

To determine the required bolt length, the value shown in Table C-2.2 should be added to the *grip* (i.e., the total thickness of all connected material, exclusive of washers). For each ASTM F436 washer that is used, add 5⁄32 in.; for each beveled washer, add 5⁄16 in. The tabulated values provide appropriate allowances for manufacturing tolerances and also provide *sufficient thread engagement* with an installed heavy-hex nut. The length determined by the use of Table C-2.2 should be adjusted to the next longer ¼-in. length increment (½-in. length increment for lengths exceeding 6 in.). A more extensive table for bolt length selection based upon these rules is available (Carter, 1996).

Pretensioned installation involves the inelastic elongation of the portion of the threaded length between the nut and the thread run-out. ASTM A490 bolts and galvanized ASTM A325 bolts possess sufficient ductility to undergo one pretensioned installation, but are not consistently ductile enough to undergo a second pretensioned installation. Black ASTM A325 bolts, however, possess sufficient ductility to undergo more than one pretensioned installation as suggested in the *Guide* (Kulak et al., 1987). As a simple rule of thumb, a black ASTM A325 bolt is suitable for reuse if the nut can be run up the threads by hand.

Table C- 2.2. Bolt Length Selection Increment

Nominal Bolt Diameter d_b, in.	To Determine the Required Bolt Length, Add to Grip, in.
$\frac{1}{2}$	$\frac{11}{16}$
$\frac{5}{8}$	$\frac{7}{8}$
$\frac{3}{4}$	1
$\frac{7}{8}$	$1\frac{1}{8}$
1	$1\frac{1}{4}$
$1\frac{1}{8}$	$1\frac{1}{2}$
$1\frac{1}{4}$	$1\frac{5}{8}$
$1\frac{3}{8}$	$1\frac{3}{4}$
$1\frac{1}{2}$	$1\frac{7}{8}$

2.4. Heavy-Hex Nuts

2.4.1. Specifications: Heavy-hex nuts shall meet the requirements of ASTM A563. The grade and finish of such nuts shall be as given in Table 2.1.

2.4.2. Geometry: Heavy-hex nut dimensions shall meet the requirements of ANSI/ASME B18.2.6.

Commentary:
Heavy-hex nuts are required by ASTM Specifications to be distinctively marked. Certain markings are mandatory. In addition to the mandatory markings, the *manufacturer* may apply additional distinguishing markings. The mandatory markings and sample optional markings are illustrated in Figure C-2.1.

Hot-dip galvanizing affects the stripping strength of the bolt-nut assembly because, to accommodate the relatively thick zinc coatings of non-uniform thickness on bolt threads, it is usual practice to hot-dip galvanize the blank nut and then to tap the nut over-size. This results in a reduction of thread engagement with a consequent reduction of the stripping strength. Only the stronger hardened nuts have adequate strength to meet ASTM thread strength requirements after over-tapping. Therefore, as specified in ASTM A325, only ASTM A563 grade DH are suitable for use as galvanized nuts. This requirement should not be overlooked if non-galvanized nuts are purchased and then sent to a local galvanizer for hot-dip galvanizing. Because the mechanical galvanizing process results in a more uniformly distributed and smooth zinc coating, nuts may be tapped over-size before galvanizing by an amount that is less than that required for the hot-dip process before galvanizing.

In earlier editions, this Specification permitted the use of ASTM A194 grade 2H nuts in the same finish as that permitted for ASTM A563 nuts in the following cases: with ASTM A325 Type 1 plain, Type 1 galvanized and Type 3 plain bolts and with ASTM A490 Type 1 plain bolts. Reference to ASTM A194 grade 2H nuts has been removed following the removal of similar reference within the ASTM A325 and A490 Specifications. However, it should be noted that ASTM A194 grade 2H nuts remain acceptable in these applications as indicated by footnote in Table 2.1, should they be available.

ASTM A563 nuts are manufactured to dimensions as specified in ANSI/ASME B18.2.6. The basic dimensions, as defined in Figure C-2.2, are shown in Table C-2.1.

2.5. Washers

Flat circular washers and square or rectangular beveled washers shall meet the requirements of ASTM F436, except as provided in Table 6.1. The type and finish of such washers shall be as given in Table 2.1.

2.6. Washer-Type Indicating Devices

The use of washer-type indicating devices is permitted as described in Sections 2.6.1 and 2.6.2.

2.6.1. Compressible-Washer-Type Direct Tension Indicators: Compressible-washer-type direct tension indicators shall meet the requirements of ASTM F959.

2.6.2. Alternative Washer-Type Indicating Devices: When approved by the *Engineer of Record*, the use of alternative washer-type indicating devices that differ from those that meet the requirements of ASTM F959 is permitted.

Detailed installation instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) The required character and frequency of pre-installation verification;
(2) The alignment of bolt holes to permit insertion of the bolt without undue damage to the threads;
(3) The placement of *fastener assemblies* in all types and sizes of holes, including placement and orientation of the alternative and regular washers;
(4) The systematic assembly of the *joint*, progressing from the most rigid part of the *joint* until the connected plies are in *firm contact*; and;
(5) The subsequent systematic pretensioning of all bolts in the *joint*, progressing from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Detailed inspection instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) Observation of the required pre-installation verification testing; and,
(2) Subsequent *routine observation* to ensure the proper use of the alternative washer-type indicating device.

2.7. Twist-Off-Type Tension-Control Bolt Assemblies

2.7.1. Specifications: Twist-off-type tension-control bolt assemblies shall meet the requirements of ASTM F1852. The *Engineer of Record* shall specify the type of bolt (Table 2.1) to be used.

2.7.2. Geometry: Twist-off-type tension-control bolt assembly dimensions shall meet the requirements of ASTM F1852. The bolt length used shall be such that the end of the bolt extends beyond or is at least flush with the outer face of the nut when properly installed.

Commentary:
It is the policy of the Research Council on Structural Connections to directly recognize only those fastener components that are manufactured to meet the requirements in an approved ASTM specification. Prior to this edition, the RCSC Specification provided for the use of ASTM A325 and A490 bolts directly and alternative-design fasteners meeting detailed requirements similar to those in Section 2.8 when approved by the *Engineer of Record*. With this edition, ASTM F1852 twist-off-type tension-control bolt assemblies are now recognized directly. Essentially, ASTM F1852 relates an ASTM A325-equivalent product to a specific method of installation that is suitable for use in all *joint* types as described in Section 8. Provision has also been retained for approval by the *Engineer of Record* of other alternative-design fasteners that meet the detailed requirements in 2.8. As an example of one such approval, the use of twist-off-type tension-control bolt assemblies with ASTM A490 mechanical properties is usually deemed acceptable.

If galvanized, ASTM F1852 twist-off-type tension-control bolt assemblies are required in ASTM F1852 to be mechanically galvanized.

While specific provisions for reuse of ASTM F1852 twist-off-type tension control bolts have not been included in this Specification, those given in Section 2.3.3 for reuse of heavy-hex structural bolts are equally applicable if the use of an alternative pretensioning method, such as the turn-of-nut pretensioning method, is practical. It is assumed that rotation of the non-turned element can be restrained.

2.8. Alternative-Design Fasteners

When approved by the *Engineer of Record*, the use of alternative-design fasteners is permitted if they:

(1) Meet the materials, manufacturing and chemical composition requirements of ASTM A325 or ASTM A490, as applicable;
(2) Meet the mechanical property requirements of ASTM A325 or ASTM A490 in full-size tests;
(3) Have a body diameter and bearing area under the bolt head and nut that is equal to or greater than those provided by a bolt and nut of the same nomi-

nal dimensions specified in Sections 2.3 and 2.4; and,

(4) Are supplied and used in the work as a *fastener assembly*.

Such alternative-design fasteners are permitted to differ in other dimensions from those of the specified *high-strength bolts* and nuts.

Detailed installation instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) The required character and frequency of pre-installation verification;
(2) The alignment of bolt holes to permit insertion of the alternative-design fastener without undue damage;
(3) The placement of *fastener assemblies* in all holes, including any washer requirements as appropriate;
(4) The systematic assembly of the *joint*, progressing from the most rigid part of the *joint* until the connected plies are in *firm contact*; and,
(5) The subsequent systematic pretensioning of all *fastener assemblies* in the *joint*, progressing from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Detailed inspection instructions shall be prepared by the *manufacturer* in a supplemental specification that is approved by the *Engineer of Record* and shall provide for:

(1) Observation of the required pre-installation verification testing; and,
(2) Subsequent *routine observation* to ensure the proper use of the alternative-design fastener.

SECTION 3. BOLTED PARTS

3.1. Connected Plies

All connected plies that are within the *grip* of the bolt and any materials that are used under the head or nut shall be steel (uncoated, coated or galvanized) as defined in Section 3.2. Compressible materials shall not be placed within the *grip* of the bolt. The slope of the surfaces of parts in contact with the bolt head and nut shall be equal to or less than 1:20 with respect to a plane that is normal to the bolt axis.

Commentary:

The presence of gaskets, insulation or any compressible materials other than the specified coatings within the *grip* would preclude the development and/or retention of the installed pretensions in the bolts, when required.

ASTM A325, F1852 and A490 bolt assemblies are ductile enough to deform to a surface with a slope that is less than or equal to 1:20 with respect to a plane normal to the bolt axis. Greater slopes are undesirable because the resultant localized bending decreases both the strength and the ductility of the bolt.

3.2. Faying Surfaces

Faying surfaces and surfaces adjacent to the bolt head and nut shall be free of dirt and other foreign material. Additionally, *faying surfaces* shall meet the requirements in Sections 3.2.1 or 3.2.2.

3.2.1. *Snug-Tightened Joints* and *Pretensioned Joints*: The *faying surfaces* of *snug-tightened joints* and *pretensioned joints* as defined in Sections 4.1 and 4.2 are permitted to be uncoated, coated with coatings of any formulation or galvanized.

Commentary:

In both *snug-tightened joints* and *pretensioned joints*, the ultimate strength is dependent upon shear transmitted by the bolts and bearing of the bolts against the connected material. It is independent of any frictional resistance that may exist on the *faying surfaces*. Consequently, since slip resistance is not an issue, the *faying surfaces* are permitted to be uncoated, coated, or galvanized without regard to the resulting slip coefficient obtained.

3.2.2. *Slip-Critical Joints*: The *faying surfaces* of *slip-critical joints* as defined in Section 4.3, including those of filler plates and finger shims, shall meet the following requirements:

(a) *Uncoated Faying Surfaces*: *Uncoated faying surfaces* shall be free of scale, except tight mill scale, and free of coatings, including inadvertent overspray, in areas closer than one bolt diameter but not less than 1 in. from the edge of any hole and in all areas within the bolt pattern.

(b) *Coated Faying Surfaces*: *Coated faying surfaces* shall first be blast cleaned and subsequently coated with a coating that is qualified in accordance with the requirements in Appendix A as a Class A or Class B coating as defined in Section 5.4. Alternatively, when approved by the *Engineer of Record*, coatings that provide a *mean slip coefficient* that differs from Class A or Class B are permitted when:

(1) The *mean slip coefficient* μ is established by testing in accordance with the requirements in Appendix A; and,
(2) The design slip resistance is determined in accordance with Section 5.4 using this coefficient, except that, for design purposes, a value of μ greater than 0.50 shall not be used.

The plies of *slip-critical joints* with *coated faying surfaces* shall not be assembled before the coating has cured for the minimum time that was used in the qualifying tests.

(c) *Galvanized Faying Surfaces*: *Galvanized faying surfaces* shall first be hot-dip galvanized in accordance with the requirements of ASTM A123 and subsequently roughened by means of hand wire brushing. Power wire brushing is not permitted. When prepared by roughening, the *galvanized faying surface* is designated as Class C for design.

Commentary:
Slip-critical joints are those *joints* that have specified *faying surface* conditions that, in the presence of the clamping force provided by pretensioned fasteners, resist a design load solely by friction and without displacement at the *faying surfaces*. Consequently, it is necessary to prepare the *faying surfaces* in a manner so that the desired slip performance is achieved.

Clean mill scale steel surfaces (Class A, see Section 5.4.1) and blast-cleaned steel surfaces (Class B, see Section 5.4.1) can be used within *slip-critical joints*. When used, it is necessary to keep the *faying surfaces* free of coatings, including inadvertent overspray.

Corrosion often occurs on uncoated blast-cleaned steel surfaces (Class B, see Section 5.4.1) due to exposure between the time of fabrication and subsequent erection. In normal atmospheric exposures, this corrosion is not detrimental and may actually increase the slip resistance of the *joint*. Yura et al. (1981) found that the Class B slip coefficient could be maintained for up to one year prior to *joint* assembly.

Polyzois and Frank (1986) demonstrated that, for plate material with thickness in the range of ⅜ in. to ¾ in., the contact pressure caused by bolt pretension is concentrated on the *faying surfaces* in annular rings around and close to the bolts. In this study, unqualified paint on the *faying surfaces* away from the edge of the bolt hole by not less than 1 in. nor the bolt diameter did not reduce the slip resistance. However, this would not likely be the case for *joints* involving thicker material, particularly those with a large number of bolts on multiple gage lines; the Table 8.1 minimum bolt pretension might not be adequate to completely flatten and pull thicker material into tight contact around every bolt. Instead, the bolt pretension would be balanced by contact pressure on the regions of the *faying surfaces* that are in contact. To account for both possibilities, it is required in this Specification that all areas between the bolts be free of coatings, including overspray, as illustrated in Figure C-3.1.

As a practical matter, the smaller coating-free area can be laid out and protected more easily using masking located relative to the bolt-hole pattern than relative to the limits of the complete area of *faying surface* contact with varying and uncertain edge distance. Furthermore, the narrow coating strip around the perimeter of the *faying surface* minimizes the required field touch-up of uncoated material outside of the *joint*.

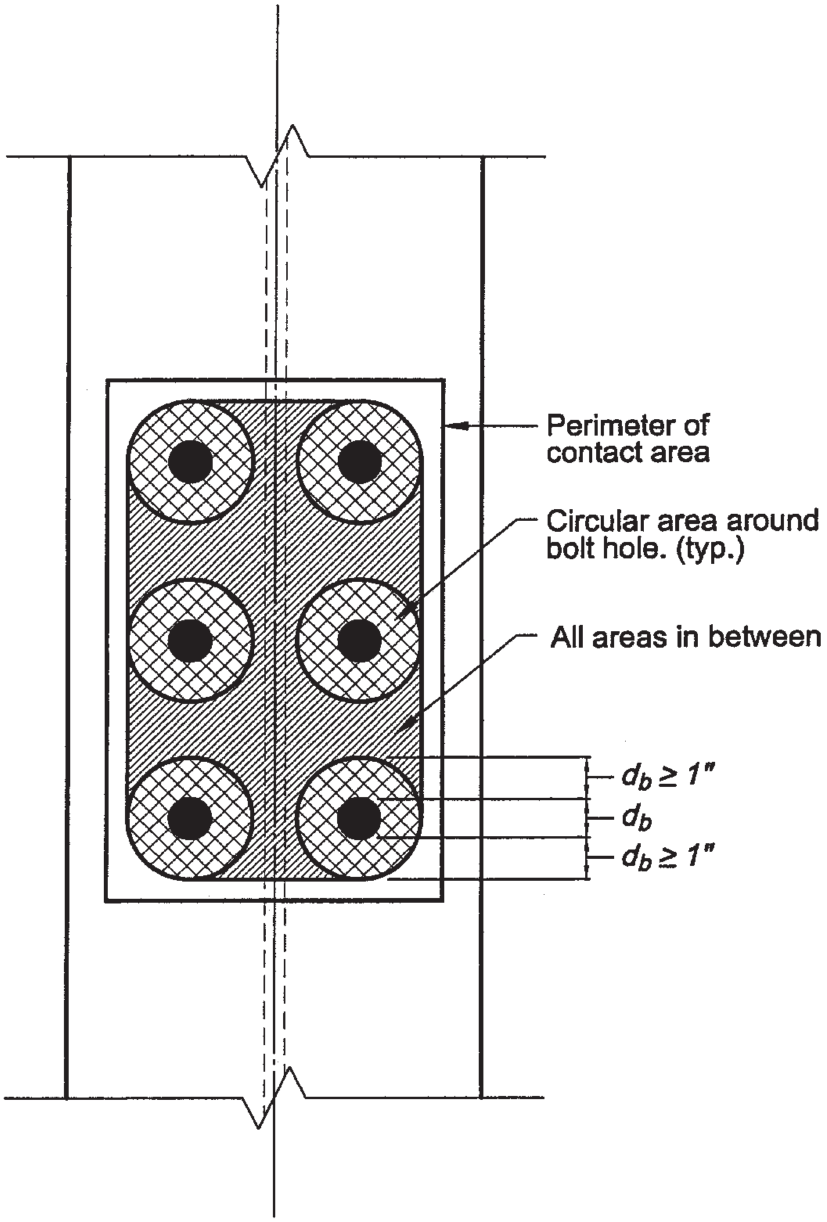


Figure C-3.1. Faying surfaces of slip-critical connections painted with unqualified paints.

Polyzois and Frank (1986) also investigated the effect of various degrees of inadvertent overspray on slip resistance. It was found that even a small amount of overspray of unqualified paint (that is, not qualified as a Class A or Class B coating) within the specified coating-free area on clean mill scale can reduce the slip resistance significantly. On blast-cleaned surfaces, however, the presence of a small amount of overspray was not as detrimental. For simplicity, this Specification requires that all overspray be prohibited from areas that are required to be free of coatings in *slip-critical joints* regardless of whether the surface is clean mill scale steel or blast-cleaned steel.

In the 1980 edition of this Specification, generic names for coatings applied to *faying surfaces* were the basis for categories of allowable working stresses in *slip-critical* (friction) *joints*. Frank and Yura (1981) demonstrated that the slip coefficients for coatings described by a generic type are not unique values for a given generic coating description or product, but rather depend also upon the type of vehicle used. Small differences in formulation from *manufacturer* to *manufacturer* or from *lot* to *lot* with a single *manufacturer* can significantly affect slip coefficients if certain essential variables within a generic type are changed. Consequently, it is unrealistic to assign coatings to categories with relatively small incremental differences between categories based solely upon a generic description.

When the *faying surfaces* of a *slip-critical joint* are to be protected against corrosion, a qualified coating must be used. A qualified coating is one that has been tested in accordance with Appendix A, the sole basis for qualification of any coating to be used in conjunction with this Specification. Coatings can be qualified as follows:

(1) As a Class A coating as defined in Section 5.4.1;
(2) As a Class B coating as defined in Section 5.4.1; or,
(3) As a coating with a *mean slip coefficient* μ other than 0.33 (Class A) but not greater than 0.50 (Class B).

Requalification is required if any essential variable associated with surface preparation, paint manufacture, application method or curing requirements is changed. See Appendix A.

Frank and Yura (1981) also investigated the effect of varying the time between coating the *faying surfaces* and assembly of the *joint* and pretensioning the bolts in order to ascertain if partially cured paint continued to cure within the assembled *joint* over a period of time. The results indicated that all curing effectively ceased at the time the *joint* was assembled and paint that was not fully cured at that time acted as a lubricant. The slip resistance of a *joint* that was assembled after a time less than the curing time used in the qualifying tests was severely reduced. Thus, the curing time prior to mating the *faying surfaces* is an essential parameter to be specified and controlled during construction.

The *mean slip coefficient* for clean hot-dip galvanized surfaces is on the order of 0.19 as compared with a factor of about 0.33 for clean mill scale. Birkemoe and Herrschaft (1970) showed that this *mean slip coefficient* can be significantly improved by treatments such as hand wire brushing or light "brush-off' grit blasting. In either case, the treatment must be controlled to achieve visible roughening or scoring. Power wire brushing is unsatisfactory because it may polish rather than roughen the surface, or remove the coating.

Field experience and test results have indicated that galvanized assemblies may continue to slip under sustained loading (Kulak et al., 1987; pp. 198-208). Tests of hot-dip galvanized *joints* subjected to sustained loading show a creep-type behavior that was not observed in short-duration or fatigue-type load application. See also the Commentary to Appendix A.

3.3. Bolt Holes

The nominal dimensions of standard, oversized, short-slotted and long-slotted holes for *high-strength bolts* shall be equal to or less than those shown in Table 3.1. Thermally cut bolt holes shall be permitted if approved by the *Engineer of Record*. For statically loaded *joints*, thermally cut surfaces need not be ground. For cyclically loaded *joints*, thermally cut surfaces shall be ground smooth.

Commentary:

The footnotes in Table 3.1 provide for slight variations in the dimensions of bolt holes from the nominal dimensions. When the dimensions of bolt holes are such that they exceed these permitted variations, the bolt hole must be treated as the next larger type.

3.3.1. Standard Holes: In the absence of approval by the *Engineer of Record* for the use of other hole types, standard holes shall be used in all plies of bolted *joints*.

Table 3.1. Nominal Bolt Hole Dimensions

Nominal Bolt Diameter, d_b, in.	Nominal Bolt Hole Dimensions[a,b], in.			
	Standard (diameter)	**Oversized (diameter)**	**Short-slotted (width × length)**	**Long-slotted (width × length)**
$\frac{1}{2}$	$\frac{9}{16}$	$\frac{5}{8}$	$\frac{9}{16} \times \frac{11}{16}$	$\frac{9}{16} \times 1\frac{1}{4}$
$\frac{5}{8}$	$\frac{11}{16}$	$\frac{13}{16}$	$\frac{11}{16} \times \frac{7}{8}$	$\frac{11}{16} \times 1\frac{9}{16}$
$\frac{3}{4}$	$\frac{13}{16}$	$\frac{15}{16}$	$\frac{13}{16} \times 1$	$\frac{13}{16} \times 1\frac{7}{8}$
$\frac{7}{8}$	$\frac{15}{16}$	$1\frac{1}{16}$	$\frac{15}{16} \times 1\frac{1}{8}$	$\frac{15}{16} \times 2\frac{3}{16}$
1	$1\frac{1}{16}$	$1\frac{1}{4}$	$1\frac{1}{16} \times 1\frac{5}{16}$	$1\frac{1}{16} \times 2\frac{1}{2}$
$\geq 1\frac{1}{8}$	$d_b + \frac{1}{16}$	$d_b + \frac{5}{16}$	$(d_b + \frac{1}{16}) \times (d_b + \frac{3}{8})$	$(d_b + \frac{1}{16}) \times (2.5 d_b)$

[a] The upper tolerance on the tabulated nominal dimensions shall not exceed $\frac{1}{32}$-in. Exception: In the width of slotted holes, gouges not more than $\frac{1}{16}$-in. deep are permitted.

[b] The slightly conical hole that naturally results from punching operations with properly matched punches and dies is acceptable.

Commentary:
The use of bolt holes $^1/_{16}$ in. larger than the bolt installed in them has been permitted since the first publication of this Specification. Allen and Fisher (1968) showed that larger holes could be permitted for *high-strength bolts* without adversely affecting the bolt shear or member bearing strength. However, the slip resistance can be reduced by the failure to achieve adequate pretension initially or by the relaxation of the bolt pretension as the highly compressed material yields at the edge of the hole or slot. The provisions for oversized and slotted holes in this Specification are based upon these findings and the additional concern for the consequences of a slip of significant magnitude if it should occur in the direction of the slot. Because an increase in hole size generally reduces the net area of a connected part, the use of oversized holes or of slotted holes is subject to approval by the *Engineer of Record*.

3.3.2. Oversized Holes: When approved by the *Engineer of Record*, oversized holes are permitted in any or all plies of *slip-critical joints* as defined in Section 4.3.

Commentary:
See the Commentary to Section 3.3.1.

3.3.3. Short-Slotted Holes: When approved by the *Engineer of Record*, short-slotted holes are permitted in any or all plies of *snug-tightened joints* as defined in Section 4.1, and *pretensioned joints* as defined in Section 4.2, provided the applied load is approximately perpendicular (between 80 and 100 degrees) to the axis of the slot. When approved by the *Engineer of Record*, short-slotted holes are permitted in any or all plies of *slip-critical joints* as defined in Section 4.3 without regard for the direction of the applied load.

Commentary:
See the Commentary to Section 3.3.1.

3.3.4. Long-Slotted Holes: When approved by the *Engineer of Record*, long-slotted holes are permitted in only one ply at any individual *faying surface* of *snug-tightened joints* as defined in Section 4.1, and *pretensioned joints* as defined in Section 4.2, provided the applied load is approximately perpendicular (between 80 and 100 degrees) to the axis of the slot. When approved by the *Engineer of Record*, long-slotted holes are permitted in one ply only at any individual *faying surface* of *slip-critical joints* as defined in Section 4.3 without regard for the direction of the applied load. Fully inserted finger shims between the *faying surfaces* of load-transmitting elements of bolted *joints* are not considered a long-slotted element of a *joint*; nor are they considered to be a ply at any individual *faying surface*.

Commentary:
See the Commentary to Section 3.3.1.

Finger shims are devices that are often used to permit the alignment and plumbing of structures. When these devices are fully and properly inserted, they do not have the same effect on bolt pretension relaxation or the *connection* performance, as do long-slotted holes in an outer ply. When fully inserted, the shim provides support around approximately 75 percent of the perimeter of the bolt in contrast to the greatly reduced area that exists with a bolt that is centered in a long slot. Furthermore, finger shims are always enclosed on both sides by the connected material, which should be effective in bridging the space between the fingers.

3.4. Burrs

Burrs that extend 1/16 in. or less above the surface are permitted to remain on the *faying surfaces* of *snug-tightened joints* as defined in Section 4.1 and *pretensioned joints* as defined in Section 4.2. Burrs that extend over 1/16 in. above the surface shall be removed from all *joints*. Burrs that would prevent solid seating of the connected plies prior to the pretensioning of *slip-critical joints* as defined in Section 4.3 shall be removed.

Commentary:

Polyzois and Yura (1985) and McKinney and Zwerneman (1993) demonstrated that the slip resistance of *joints* was either unchanged or slightly improved by the presence of burrs. Therefore, small (1/16 in. or less) burrs that do not prevent solid seating of the connected parts need not be removed. On the other hand, parallel tests in the same program demonstrated that large burrs (over 1/16 in.) could cause a small increase in the required nut rotation from the snug-tight condition to achieve the specified pretension with the turn-of—nut pretensioning method. In the interest of simplicity, this Specification requires that all large burrs be removed.

SECTION 4. JOINT TYPE

For *joints* with fasteners that are loaded in shear or combined shear and tension, the *Engineer of Record* shall specify the *joint* type in the contract documents as snug-tightened, pretensioned or slip-critical. For *slip-critical joints*, the required class of slip resistance in accordance with Section 5.4 shall also be specified. For *joints* with fasteners that are loaded in tension only, the *Engineer of Record* shall specify the *joint* type in the contract documents as snug-tightened or pretensioned. Table 4.1 summarizes the applications and requirements of the three *joint* types.

Table 4.1. Summary of Applications and Requirements for Bolted Joints

Load Transfer	Application	Joint Type[a,b]	Faying Surface Prep.?	Install per Section	Inspect per Section	Arbitrate per Section 10?
Shear only	Resistance to shear load by shear/bearing	ST	No	8.1	9.1	No
	Resistance to shear by shear/bearing. Bolt pretension is required, but for reasons other than slip resistance.	PT	No	8.2	9.2	No
	Shear-load resistance by friction on faying surfaces is required.	SC	Yes[d]	8.2	9.3	If required to resolve dispute
Combined shear and tension	Resistance to shear load by shear/bearing. Tension load is static only.[c]	ST	No	8.1	9.1	No
	Resistance to shear by shear/bearing. Bolt pretension is required, but for reasons other than slip resistance.	PT	No	8.2	9.2	If required to resolve dispute
	Shear-load resistance by friction on faying surfaces is required.	SC	Yes[d]	8.2	9.3	If required to resolve dispute
Tension only	Static loading only.[c]	ST	No	8.1	9.1	No
	All other conditions of tension-only loading.	PT	No	8.2	9.2	If required to resolve dispute

[a] Under Joint Type: ST = snug-tightened, PT = pretensioned and SC = slip-critical; See Section 4.

[b] See Sections 4 and 5 for the design requirements for each joint type.

[c] Per Section 4.2, the use of ASTM A490 bolts in snug-tightened joints with tensile loads is not permitted.

[d] See Section 3.2.2.

Commentary:
When first approved by the Research Council on Structural Connections, in January, 195l, the "Specification for Assembly of Structural Joints Using High-Strength Bolts" merely permitted the substitution of a like number of ASTM A325 bolts for hot driven ASTM A141[2] steel rivets of the same nominal diameter. Additionally, it was required that all bolts be pretensioned and that all *faying surfaces* be free of paint; hence, satisfying the requirements for a *slip-critical joint* by the present-day definition. As revised in 1954, the omission of paint was required to apply only to "*joints* subject to stress reversal, impact or vibration, or to cases where stress redistribution due to *joint* slippage would be undesirable." This relaxation of the earlier provision recognized the fact that, in many applications, movement of the connected parts that brings the bolts into bearing against the sides of their holes is in no way detrimental. Bolted *joints* were then designated as "bearing type", "friction type" or "direct tension". With the 1985 edition of this Specification, these designations were changed to "shear/bearing", "slip-critical" and "direct tension", respectively, and snug-tightened installation was permitted for many *shear/bearing joints*. With this edition of this Specification, *snug-tightened joints* are also permitted for qualified applications involving ASTM A325 bolts in direct tension.

If non-pretensioned bolts are used in the type of *joint* that places the bolts in shear, load is transferred by shear in the bolts and bearing stress in the connected material. At the ultimate limit state, failure will occur by shear failure of the bolts, by bearing failure of the connected material or by failure of the member itself. On the other hand, if pretensioned bolts are used in such a *joint*, the frictional force that develops between the connected plies will initially transfer the load. Until the frictional force is exceeded, there is no shear in the bolts and no bearing stress in the connected components. Further increase of load places the bolts into shear and against the connected material in bearing, just as was the case when non-pretensioned bolts were used. Since it is known that the pretension in bolts will have been dissipated by the time bolt shear failure takes place (Kulak et al., 1987; p. 49), the ultimate limit state of a pretensioned bolted *joint* is the same as an otherwise identical *joint* that uses non-pretensioned bolts.

Because the consequences of slip into bearing vary from application to application, the determination of whether a *joint* can be designated as snug-tightened or as pre-tensioned or rather must be designated as slip-critical is best left to judgment and a decision on the part of the *Engineer of Record*. In the case of *joints* with three or more bolts in holes with only a small clearance, the freedom to slip generally does not exist. It is probable that normal fabrication tolerances and erection procedures are such that one or more bolts are in bearing even before additional load is applied. Such is the case for standard holes and for slotted holes loaded transverse to the axis of the slot.

Joints that are required to be *slip-critical joints* include:

(1) Those cases where slip movement could theoretically exceed an amount deemed by the *Engineer of Record* to affect the serviceability of the structure or through excessive distortion to cause a reduction in strength or stability, even though the resistance to fracture of the *connection* and yielding of the member may be adequate; and,

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[2] ASTM A141 (discontinued in 1967) became identified as A502 Grade 1 (discontinued 1999).

(2) Those cases where slip of any magnitude must be prevented, such as in *joints* subject to significant load reversal and *joints* between elements of built-up compression members in which any slip could cause a reduction of the flexural stiffness required for the stability of the built-up member.

In this Specification, the provisions for the design, installation and inspection of bolted *joints* are dependent upon the type of *joint* that is specified by the *Engineer of Record*. Consequently, it is required that the *Engineer of Record* identify the *joint* type in the contract documents.

4.1. Snug-Tightened Joints

Except as required in Sections 4.2 and 4.3, *snug-tightened joints* are permitted.

Bolts in *snug-tightened joints* shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2 and 5.3, installed in accordance with Section 8.1 and inspected in accordance with Section 9.1. As indicated in Section 4 and Table 4.1, requirements for *faying surface* condition shall not apply to *snug-tightened joints*.

Commentary:

Recognizing that the ultimate strength of a *connection* is independent of the bolt pretension and slip movement, there are numerous practical cases in the design of structures where, if slip occurs, it will not be detrimental to the serviceability of the structure. Additionally, there are cases where slip of the *joint* is desirable to permit rotation in a *joint* or to minimize the transfer of moment. To provide for these cases while at the same time making use of the shear strength of *high-strength bolts*, *snug-tightened joints* are permitted.

The maximum amount of slip that can occur in a *joint* is, theoretically, equal to twice the hole clearance. In practical terms, it is observed in laboratory and field experience to be much less; usually, about one-half the hole clearance. Acceptable inaccuracies in the location of holes within a pattern of bolts usually cause one or more bolts to be in bearing in the initial, unloaded condition. Furthermore, even with perfectly positioned holes, the usual method of erection causes the weight of the connected elements to put some of the bolts into direct bearing at the time the member is supported on loose bolts and the lifting crane is unhooked. Additional loading in the same direction would not cause additional *joint* slip of any significance.

With this edition of this Specification, *snug-tightened joints* are also permitted for statically loaded applications involving ASTM A325 bolts and ASTM F1852 twist-off-type tension-control bolt assemblies in direct tension. However, snug-tightened installation is not permitted for these fasteners in applications involving non-static loading, nor for applications involving ASTM A490 bolts.

4.2. Pretensioned Joints

Pretensioned joints are only required in the following applications:

(1) *Joints* in which fastener pretension is required in the specification or code that invokes this Specification;
(2) *Joints* that are subject to significant load reversal;
(3) *Joints* that are subject to fatigue load with no reversal of the loading direction;
(4) *Joints* with ASTM A325 or F1852 bolts that are subject to tensile fatigue; and,
(5) *Joints* with ASTM A490 bolts that are subject to tension or combined shear and tension, with or without fatigue.

Bolts in *pretensioned joints* subject to shear shall be designed in accordance with the applicable provisions of Sections 5.1 and 5.3, installed in accordance with Section 8.2 and inspected in accordance with Section 9.2. Bolts in *pretensioned joints* subject to tension or combined shear and tension shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2, 5.3 and 5.5, installed in accordance with Section 8.2 and inspected in accordance with Section 9.2. As indicated in Section 4 and Table 4.1, requirements for *faying surface* condition shall not apply to *pretensioned joints*.

Commentary:
Under the provisions of some other specifications, certain shear *connections* are required to be pretensioned, but are not required to be slip-critical. Several cases are given, for example, in AISC LRFD Specification Section J1.11 (AISC, 1999) wherein certain bolted *joints* in bearing *connections* are to be pretensioned regardless of whether or not the potential for slip is a concern. The AISC Specification requires that *joints* be pretensioned in the following circumstances:

(1) Column splices in buildings with high ratios of height to width;
(2) *Connections* of members that provide bracing to columns in tall buildings;
(3) Various *connections* in buildings with cranes over 5-ton capacity; and,
(4) *Connections* for supports of running machinery and other sources of impact or stress reversal.

When pretension is desired for reasons other than the necessity to prevent slip, a *pretensioned joint* should be specified in the contract documents.

4.3. Slip-Critical Joints

Slip-critical joints are only required in the following applications involving shear or combined shear and tension:

(1) *Joints* that are subject to fatigue load with reversal of the loading direction;
(2) *Joints* that utilize oversized holes;
(3) *Joints* that utilize slotted holes, except those with applied load approximately normal (within 80 to 100 degrees) to the direction of the long dimension of the slot; and,
(4) *Joints* in which slip at the *faying surfaces* would be detrimental to the performance of the structure.

Bolts in *slip-critical joints* shall be designed in accordance with the applicable provisions of Sections 5.1, 5.2, 5.3, 5.4 and 5.5, installed in accordance with Section 8.2 and inspected in accordance with Section 9.3.

Commentary:

In certain cases, slip of a bolted *joint* in shear under service loads would be undesirable or must be precluded. Clearly, *joints* that are subject to reversed fatigue load must be slip-critical since slip may result in back-and-forth movement of the *joint* and the potential for accelerated fatigue failure. Unless slip is intended, as desired in a sliding expansion *joint*, slip in *joints* with long-slotted holes that are parallel to the direction of the applied load might be large enough to invalidate structural analyses that are based upon the assumption of small displacements.

For *joints* subject to fatigue load with respect to shear of the bolts that does not involve a reversal of load direction, there are two alternatives for fatigue design. The designer can provide either a *slip-critical joint* that is proportioned on the basis of the applied stress range on the gross section, or a *pretensioned joint* that is proportioned on the basis of applied stress range on the net section.

SECTION 5. LIMIT STATES IN BOLTED JOINTS

The design shear strength and design tensile strength of bolts shall be determined in accordance with Section 5.1. The interaction of combined shear and tension on bolts shall be limited in accordance with Section 5.2. The design bearing strength of the connected parts at bolt holes shall be determined in accordance with Section 5.3. Each of these *design strengths* shall be equal to or greater than the *required strength*. The axial load in bolts that are subject to tension or combined shear and tension shall be calculated with consideration of the effects of the externally applied tensile load and any additional tension resulting from *prying action* produced by deformation of the connected parts.

When slip resistance is required at the *faying surfaces* subject to shear or combined shear and tension, slip resistance shall be checked at either the factored-load level or service-load level, at the option of the *Engineer of Record*. When slip of the *joint* under factored loads would affect the ability of the structure to support the factored loads, the *design strength* determined in accordance with Section 5.4.1 shall be equal to or greater than the *required strength*. When slip resistance under service loads is the design criterion, the strength determined in accordance with Section 5.4.2 shall be equal to or greater than the effect of the service loads. In addition, slip-critical connections must meet the strength requirements to resist the factored loads as shear/bearing joints. Therefore, the strength requirements of Sections 5.1, 5.2 and 5.3 shall also be met.

When bolts are subject to cyclic application of axial tension, the stress determined in accordance with Section 5.5 shall be equal to or greater than the stress due to the effect of the service loads, including any additional tension resulting from prying action produced by deformation of the connected parts.

Commentary:

This section of the Specification provides the design requirements for *high-strength bolts* in bolted *joints*. However, this information is not intended to provide comprehensive coverage of the design of *high-strength bolted connections*. Other design considerations of importance to the satisfactory performance of the connected material, such as block shear rupture, shear lag, *prying action* and *connection* stiffness and its effect on the performance of the structure, are beyond the scope of this Specification and Commentary.

The design of bolted *joints* that transmit shear requires consideration of the shear strength of the bolts and the bearing strength of the connected material. If such *joints* are designated as *slip-critical joints*, the slip resistance must also be checked. This serviceability check can be made at the factored-load level (Section 5.4.1) or at the service-load level (Section 5.4.2). Regardless of which load level is selected for the check of slip resistance, the prevention of slip in the service-load range is the design criterion.

Parameters that influence the shear strength of bolted *joints* include:

(1) Geometric parameters – the ratio of the net area to the gross area of the connected parts, the ratio of the net area of the connected parts to the total shear-resisting area of the bolts and the length of the *joint*; and,

(2) Material parameter – the ratio of the yield strength to the tensile strength of the connected parts.

Using both mathematical models and physical testing, it was possible to study the influences of these parameters (Kulak et al., 1987; pp. 89-116 and 126-132). These showed that, under the rules that existed at that time the longest (and often the most important) *joints* had the lowest factor of safety, about 2.0 based on ultimate strength.

In general, bolted *joints* that are designed in accordance with the provisions of this Specification will have a higher reliability than will the members they connect. This occurs primarily because the resistance factors used in limit states for the design of bolted *joints* were chosen to provide a reliability higher than that used for member design. Additionally, the controlling strength limit state in the structural member, such as yielding or deflection, is usually reached well before the strength limit state in the *connection*, such as bolt shear strength or bearing strength of the connected material. The installation requirements vary with *joint* type and influence the behavior of the *joints* within the service-load range, however, this influence is ignored in all strength calculations. Secondary tensile stresses that may be produced in bolts in *shear/bearing joints*, such as through the flexing of double-angle *connections* to accommodate the simple-beam end rotation, need not be considered.

It is sometimes necessary to use *high-strength bolts* and fillet welds in the same *connection*, particularly as the result of remedial work. When these fastening elements act in the same shear plane, the combined strength is a function of whether the bolts are snug-tightened or pretensioned, the location of the bolts relative to the holes in which they are located and the orientation of the fillet welds. The fillet welds can be parallel or transverse to the direction of load. Recent work (Manuel and Kulak, 1999) can be used to calculate the *design strength* of such *joints*.

5.1. Design Shear and Tensile Strengths

Shear and tensile strengths shall not be reduced by the installed bolt pretension. For *joints*, the design shear and tensile strengths shall be taken as the sum of the strengths of the individual bolts.

The design strength in shear or the design strength in tension for an ASTM A325, A490 or F1852 bolt is ϕR_n, where $\phi = 0.75$ and:

$$R_n = F_n A_b \qquad \text{(Equation 5.1)}$$

where

R_n = nominal strength (shear strength per shear plane or tensile strength) of a bolt, kips;

F_n = nominal strength per unit area from Table 5.1 for the appropriate applied load conditions, ksi, adjusted for the presence of fillers as fillers as required below, and,

A_b = cross-sectional area based upon the nominal diameter of bolt, in.^{2}

When a bolt that carries load passes through fillers or shims in a shear plane that are equal to or less than 1/4-in. thick, F_n from Table 5.1 shall be used without reduction. When a bolt that carries load passes through fillers or shims that are greater than 1/4-in. thick, they shall be designed in accordance with one of the following procedures:

Table 5.1. Nominal Strength per Unit Area of Bolts

Applied Load Condition		**Nominal Strength per Unit Area F_n, ksi**	
		ASTM A325 or F1852 Bolt	**ASTM A490 Bolt**
Tension[a]	**Static**	90	113
	Fatigue	See Section 5.5	
Shear[a,b]	**Threads included in shear plane**	48	60
	Threads excluded from shear plane	60	75

[a] Except as required in Section 5.2.

[b] In shear *connections* that transmit axial force and have length between extreme bolts measured parallel to the line of force exceeds 50 in., tabulated values shall be reduced by 20 percent.

(1) For fillers or shims that are equal to or less than ¾ in. thick, F_n from Table 5.1 shall be multiplied by the factor [1 - 0.4(t' - 0.25)], where t' is the total thickness of fillers or shims, in., up to ¾ in.;

(2) The fillers or shims shall be extended beyond the *joint* and the filler or shim extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers or shims;

(3) The size of the *joint* shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

(4) The *joint* shall be designed as a *slip-critical joint*. The slip resistance of the *joint* shall not be reduced for the presence of fillers or shims.

Commentary:

The nominal shear and tensile strengths of ASTM A325, F1852 and A490 bolts are given in Table 5.1. These values are based upon the work of a large number of researchers throughout the world, as reported in the *Guide* (Kulak et al., 1987). The *design strength* equals the *nominal strength* multiplied by a resistance factor ϕ. On average, the *design strengths* result in bolted *joint* designs that are approximately equivalent to those provided under the allowable stress rules given in the 1980 edition of this Specification.

The nominal shear strength is based upon the observation that the shear strength of a single *high-strength bolt* is about 0.62 times the tensile strength of that bolt (Kulak et al., 1987; pp. 44-50). However, in lap splices transmitting axial force between members with more than two bolts in the line of force, non-uniform deformation of the connected material between fasteners causes a non-uniform distribution of the shear force to the bolts. Consequently, the strength of the *joint* decreases in terms of the average strength of all the bolts in the *joint* as the *joint* length increases (Kulak et al., 1987; pp. 99-104). Rather than provide a decreasing function that reflects this decrease in average fastener strength with *joint* length, a single reduction factor of 0.80 is applied to the 0.62 multiplier. This accommodates bolts in all *joints* up to 50 in. in length without seriously affecting the economy of very short *joints*. As noted in Footnote b in Table 5.1, the average shear strength of bolts in *joints* longer than 50 in. in length must be further reduced by 20 percent. Note that this reduction does not apply in cases when the distribution of force is essentially uniform along the *joint*, such as the bolted *joints* in a shear *connection* at the end of a deep plate girder.

The average ratio of nominal shear strength for bolts with threads included in the shear plane to the nominal shear strength for bolts with threads excluded from the shear plane is 0.83 with a standard deviation of 0.03 (Frank and Yura, 1981). Conservatively, a reduction factor of 0.80 is used to account for the reduction in shear strength for a bolt with threads included in the shear plane but calculated with the area corresponding to the nominal bolt diameter. The case of a bolt in double shear with a non-threaded section in one shear plane and a threaded section in the other shear plane is not covered in this Specification for two reasons. First, the manner in which load is shared between these two dissimilar shear areas is uncertain. Second, the detailer's lack of certainty as to the orientation of the bolt placement might leave both shear planes in the threaded section. Thus, if threads are included in one shear plane, the conservative assumption is made that threads are included in all shear planes.

The tensile strength of a *high-strength bolt* is the product of its ultimate tensile strength (per unit area) and some area through the threaded portion. This area, called the tensile stress area, is a derived quantity that is a function of the relative thread size and pitch. For the usual sizes of structural bolts, it is about 75 percent of the nominal cross-sectional area of the bolt. Hence, the nominal tensile strengths per unit area given in Table 5.1 are 0.75 times the tensile strength of the bolt material. According to Equation 5.1, the nominal area of the bolt is then used to calculate the *design strength* in tension. The *nominal strengths* so-calculated are intended to form the basis for comparison with the externally applied bolt tension plus any additional tension that results from *prying action* that is produced by deformation of the connected elements.

If pretensioned bolts are used in a *joint* that loads the bolts in tension, the question arises as to whether the pretension and the applied tension are additive. Because the compressed parts are being unloaded during the application of the external tensile force, the increase in bolt tension is minimal until the parts separate (Kulak et al., 1987; pp. 263-266). Thus, there will be little increase in bolt force above the pretension load under service loads. After the parts separate, the bolt acts as a tension member, as expected, and its *design strength* is that given in Equation 5.1 multiplied by the resistance factor ϕ.

Pretensioned bolts have torsion present during the installation process. Once the installation is completed, any residual torsion is quite small and will disappear entirely when the fastener is loaded to the point of plate separation. Hence, there is no question of torsion—tension interaction when considering the ultimate tensile strength of a *high-strength bolt* (Kulak et al., 1987; pp. 41-47).

When required, pretension is induced in a bolt by imposing a small axial elongation during installation, as described in the Commentary to Section 8. When the *joint* is subsequently loaded in shear, tension or combined shear and tension, the bolts will undergo significant deformations prior to failure that have the effect of overriding the small axial elongation that was introduced during installation and, thereby, removing the pretension. Measurements taken in laboratory tests confirm that the pretension that would be sustained if the applied load were removed is essentially zero before the bolt fails in shear (Kulak et al., 1987; pp. 93-94). Thus, the shear and tensile strengths of a bolt are not affected by the presence of an initial pretension in the bolt.

See also the Commentary to Section 5.5.

5.2. Combined Shear and Tension

When combined shear and tension loads are transmitted by an ASTM A325, A490 or F1852 bolt, the ultimate limit-state interaction shall be:

$$\left[\frac{T_u}{(\phi R_n)_t}\right]^2 + \left[\frac{V_u}{(\phi R_n)_v}\right]^2 \le 1 \qquad \text{(Equation 5.2)}$$

where

T_u = *required strength* in tension (factored tensile load) per bolt, kips;

V_u = *required strength* in shear (factored shear load) per bolt, kips;

$(\phi R_n)_t$ = *design strength* in tension determined in accordance with Section 5.1, kips; and,

$(\phi R_n)_v$ = *design strength* in shear determined in accordance with Section 5.1, kips

Commentary:

When both shear forces and tensile forces act on a bolt, the interaction can be conveniently expressed as an elliptical solution (Chesson et al., 1965) that includes the elements of the bolt acting in shear alone and the bolt acting in tension alone. Although the elliptical solution provides the best estimate of the strength of bolts subject to combined shear and tension and is thus used in this Specification, the nature of the elliptical solution is such that it can be approximated conveniently using three straight lines (Carter et al., 1997). Earlier editions of this specification have used such linear representations for the convenience of design calculations. The elliptical interaction equation in effect shows that, for design purposes, significant interaction does not occur until either force component exceeds 20 percent of the limiting strength for that component.

5.3. Design Bearing Strength at Bolt Holes

For *joints*, the design bearing strength shall be taken as the sum of the strengths of the connected material at the individual bolt holes.

The design bearing strength of the connected material at a standard bolt hole, oversized bolt hole, short-slotted bolt hole independent of the direction of loading or long-slotted bolt hole with the slot parallel to the direction of the bearing load is ϕR_n, where $\phi = 0.75$ and:

(1) when deformation of the bolt hole at service load is a design consideration;

$$R_n = 1.2 L_c t F_u \le 2.4 d_b t F_u \qquad \text{(Equation 5.3)}$$

(2) when deformation of the bolt hole at service load is not a design consideration;

$$R_n = 1.5L_c t F_u \leq 3 d_b t F_u \qquad \text{(Equation 5.4)}$$

The design bearing strength of the connected material at a long-slotted bolt hole with the slot perpendicular to the direction of the bearing load is ϕR_n, where ϕ = 0.75 and:

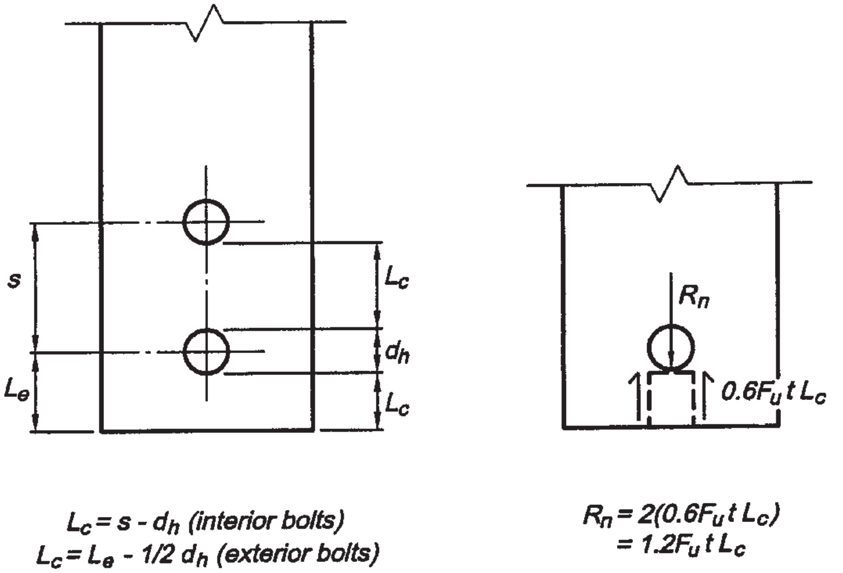
$$R_n = L_c t F_u \leq 2 d_b t F_u \qquad \text{(Equation 5.5)}$$

In Equations 5.3, 5.4 and 5.5,

R_n = *nominal strength* (bearing strength of the connected material), kips;
F_u = specified minimum tensile strength (per unit area) of the connected material, ksi;
L_c = clear distance, in the direction of load, between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.;
d_b = nominal diameter of bolt, in.; and,
t = thickness of the connected material, in.

Commentary:
The contact pressure at the interface between a bolt and the connected material can be expressed as a bearing stress on the bolt or on the connected material. The connected material is always critical. For simplicity, the bearing area is expressed as the bolt diameter times the thickness of the connected material in bearing. The governing value of the bearing stress has been determined from extensive experimental research and a further limitation on strength was derived from the case of a bolt at the end of a tension member or near another fastener.

The design equations are based upon the models presented in the *Guide* (Kulak et al., 1987; pp. 141-143), except that the clear distance to another hole or edge is used in the Specification formulation rather than the bolt spacing or end distance as used in the *Guide* (see Figure C-5.1). Equation 5.3 is derived from tests (Kulak et al., 1987; pp. 112-116) that showed that the total elongation, including local bearing deformation, of a standard hole that is loaded to obtain the ultimate strength equal to $3d_b tF_u$ in Equation 5.4 was on the order of the diameter of the bolt. This apparent hole elongation results largely from bearing deformation of the material that is immediately adjacent to the bolt. The lower value of $2.4d_b tF_u$ in Equation 5.3 provides a bearing strength limit-state that is attainable at reasonable deformation (¼ in.). Strength and deformation limits were thus used to jointly evaluate bearing strength test results for design.



(a) Dimensions

(b) Strength formulation (per bolt)

Figure. C-5.1. Bearing strength formulation.

When long-slotted holes are oriented with the long dimension perpendicular to the direction of load, the bending component of the deformation in the material between adjacent holes or between the hole and the edge of the plate is increased. The nominal bearing strength is limited to $2d_b tF_u$, which again provides a bearing strength limit-state that is attainable at reasonable deformation.

The design bearing strength has been expressed as that of a single bolt, although it is really that of the connected material that is immediately adjacent to the bolt. In calculating the design bearing strength of a connected part, the total bearing strength of the connected part can be taken as the sum of the bearing strengths of the individual bolts.

5.4. Design Slip Resistance

5.4.1. At the Factored-Load Level: The design slip resistance is ϕR_n, where ϕ is as defined below and:

$$R_n = \mu D_u T_m N_b \left(1 - \frac{T_u}{D_u T_m N_b}\right) \qquad \text{(Equation 5.6)}$$

where

ϕ = 1.0 for standard holes
= 0.85 for oversized and short-slotted holes
= 0.70 for long-slotted holes perpendicular to the direction of load
= 0.60 for long-slotted holes parallel to the direction of load;

R_n = *nominal strength* (slip resistance) of a slip plane, kips;

μ = *mean slip coefficient* for Class A, B or C *faying surfaces*, as applicable, or as established by testing in accordance with Appendix A (see Section 3.2.2(b))

= 0.33 for Class A *faying surfaces* (uncoated clean mill scale steel surfaces or surfaces with Class A coatings on blast-cleaned steel)

= 0.50 for Class B surfaces (uncoated blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel

= 0.35 for Class C surfaces (roughened hot-dip galvanized surfaces);

D_u = 1.13, a multiplier that reflects the ratio of the mean installed bolt pretension to the specified minimum bolt pretension T_m; the use of other values of D_u shall be approved by the *Engineer of Record;*

T_m = specified minimum bolt pretension (for *pretensioned joints* as specified in Table 8.1), kips;

N_b = number of bolts in the *joint*; and

T_u = *required strength* in tension (tensile component of applied factored load for combined shear and tension loading), kips

= zero if the *joint* is subject to shear only

5.4.2. At the Service-Load Level: The service-load slip resistance is ϕR_s, where ϕ is as defined in Section 5.4.1 and:

$$R_s = \mu D T_m N_b \left(1 - \frac{T}{D T_m N_b}\right) \qquad \text{(Equation 5.7)}$$

where

D = 0.80, a slip probability factor that reflects the distribution of actual slip coefficient values about the mean, the ratio of mean installed bolt pretension to the specified minimum bolt pretension, T_m, and a slip probability level; the use of other values of D must be approved by the *Engineer of Record*

T = applied service load in tension (tensile component of applied service load for combined shear and tension loading), kips

= zero if the *joint* is subject to shear only

and all other variables are as defined for Equation 5.6.

Commentary:

The design check for slip resistance can be made either at the factored-load level (Section 5.4.1) or at the service-load level (Section 5.4.2). These alternatives are based upon different design philosophies, which are discussed below. They have been calibrated to produce results that are essentially the same. The factored-load level approach is provided for the expedience of only working with factored loads. Irrespective of the approach, the limit state is based upon the prevention of slip at service-load levels.

If the factored-load provision is used, the *nominal strength* R_n represents the mean resistance, which is a function of the *mean slip coefficient* μ and the specified minimum bolt pretension (clamping force) T_m. The 1.13 multiplier in Equation 5.6 accounts for the expected 13 percent higher mean value of the installed bolt pretension provided by the calibrated wrench pretensioning method compared to the specified minimum bolt pretension T_m used in the calculation. In the absence of other field test data, this value is used for all methods.

If the service-load approach is used, a probability of slip is identified. It implies that there is 90 percent reliability that slip will not occur at the calculated slip load if the calibrated wrench pretensioning method is used, or that there is 95 percent reliability that slip will not occur at the calculated slip load if the turn-of-nut pretensioning method is used. The probability of loading occurrence was not considered in developing these slip probabilities (Kulak et al., 1987; pg. 135).

For most applications, the assumption that the slip resistance at each fastener is equal and additive with that at the other fasteners is based on the fact that all locations must develop the slip force before a total *joint* slip can occur at that plane. Similarly the forces developed at various slip planes do not necessarily develop simultaneously, but one can assume that the full slip resistances must be mobilized at each plane before full *joint* slip can occur. Equations 5.6 and 5.7 are formulated for the general case of a single slip plane. The total slip resistance of a *joint* with multiple slip planes can be calculated as that for a single slip plane multiplied by the number of slip planes.

Only the *Engineer of Record* can determine whether the potential slippage of a *joint* is critical at the service-load level as a serviceability consideration only or whether slippage could result in distortions of the frame such that the ability of the frame to resist the factored loads would be reduced. The following comments reflect the collective thinking of the Council and are provided as guidance and an indication of the intent of the Specification. See also the Commentary to Sections 4.2 and 4.3.

(1) If *joints* with standard holes have only one or two bolts in the direction of the applied load, a small slip may occur. In this case, *joints* subject to vibration should be proportioned to resist slip at the service-load level.

(2) In built-up compression members, such as double-angle struts in trusses, a small relative slip between the elements especially at the end *connections* can increase the effective length of the combined cross-section to that of the individual components and significantly reduce the compressive strength of the strut. Therefore, the *connection* between the elements at the ends of built-up members should be checked at the factored-load level, whether or not a *slip-critical joint* is required for serviceability. As given by Sherman and Yura (1998), the required slip resistance is $0.008P_uLQ/I$, where P_u is the axial compressive force in the built-up member, kips, L is the total length of the built-up member, in., Q is the first moment of area of one component about the axis of buckling of the built-up member, in.^{3}, and I is the moment of inertia of the built-up member about the axis of buckling, in.^{4}

(3) In *joints* with long-slotted holes that are parallel to the direction of the applied load, the designer has two alternatives. The *joint* can be designed to prevent slip in the service-load range using either the factored-load-level provision in Section 5.4.1 or the service-load-level provision in Section 5.4.2. In either case, however, the effect of the factored loads acting on the deformed structure (deformed by the maximum amount of slip in the long slots at all locations) must be included in the structural analysis.

(4) In *joints* subject to fatigue, design should be based upon service-load criteria and the design slip resistance of Section 5.4.2 because fatigue is a function of the service load performance rather than that of the factored load.

Extensive data developed through research sponsored by the Council and others during the past twenty years has been statistically analyzed to provide improved information on slip probability of *joints* in which the bolts have been pretensioned to the requirements of Table 8.1. Two variables, the *mean slip coefficient* of the *faying surfaces* and the bolt pretension, were found to affect the slip resistance of *joints*. Field studies (Kulak and Birkemoe, 1993) of installed bolts in various structural applications indicate that the Table 8.1 pretensions have been achieved as anticipated in the laboratory research.

An examination of the slip-coefficient data for a wide range of surface conditions indicates that the data are distributed normally and the standard deviation is essentially the same for each surface condition class. This means that different reduction factors should be applied to classes of surfaces with different *mean slip coefficients*—the smaller the mean value of the coefficient of friction, the smaller (more severe) the appropriate reduction factor—to provide equivalent reliability of slip resistance.

The bolt clamping force data indicate that bolt pretensions are distributed normally for each pretensioning method. However, the data also indicate that the mean value of the bolt pretension is different for each method. As noted previously, if the calibrated wrench method is used to pretension ASTM A325 bolts, the mean value of bolt pretension is about 1.13 times the specified minimum pretension in Table 8.1. If the turn-of-nut pretensioning method is used, the mean pretension is about 1.35 times the specified minimum pretension for ASTM A325 bolts and about 1.26 for ASTM A490 bolts.

The combined effects of the variability of the *mean slip coefficient* and bolt pretension have been accounted for approximately in the single value of the slip probability factor D in the equation for nominal slip resistance in Section 5.4.2. This implies 90 percent reliability that slip will not occur if the calibrated wrench pretensioning method is used and 95 percent reliability if the turn-of-nut pretensioning method is used. For values of D that are appropriate for other *mean slip coefficients* and slip probabilities, refer to the *Guide* (Kulak et al., 1987; pg. 135). The values given therein are suitable for direct substitution into the formula for slip resistance in Section 5.4.2.

The calibrated wrench installation method targets a specific bolt pretension, which is 5 percent greater than the specified minimum value given in Table 8.1. Thus, regardless of the actual strength of production bolts, this target value is unique for a given fastener grade. On the other hand, the turn-of-nut installation method imposes an elongation on the fastener. Consequently, the inherent strength of the bolts being installed will be reflected in the resulting pretension because this elongation will bring the fastener to its proportional limit under combined torsion and tension. As a result of these differences, the mean value and nature of the frequency distribution of pretensions for the two installation methods differ. Turn-of-nut installations result in higher mean levels of pretension than do calibrated wrench installations. These differences were taken into account when the design criteria for *slip-critical joints* were developed.

Statistical information on the pretension characteristics of bolts installed in the field using direct tension indicators and twist-off-type tension-control bolts is limited.

In any of the foregoing installation methods, it can be expected that a portion of the bolt assembly (the threaded portion of the bolt within the *grip* length and/or the engaged threads of the nut and bolt) will reach the inelastic region of behavior. This permanent distortion has no undesirable effect on the subsequent performance of the bolt.

Because of the greater likelihood that significant deformation can occur in *joints* with oversized or slotted holes, lower values of design slip resistance are provided for *joints* with these hole types through a modification of the resistance factor ϕ. For the case of long-slotted holes, even though the slip load is the same for loading transverse or parallel to the axis of the slot, the value for loading parallel to the axis has been further reduced, based upon judgment, in recognition of the greater consequences of slip.

Although the design philosophy for *slip-critical joints* presumes that they do not slip into bearing when subject to loads in the service range, it is mandatory that *slip-critical joints* also meet the requirements of Sections 5.1, 5.2 and 5.3. Thus, they must meet the strength requirements to resist the factored loads as *shear/bearing joints*.

Section 3.2.2(b) permits the *Engineer of Record* to authorize the use of *faying surfaces* with a *mean slip coefficient* μ that is less than 0.50 (Class B) and other than 0.33 (Class A) This authorization requires that the following restrictions are met:

(1) The *mean slip coefficient* μ must be determined in accordance with Appendix A; and,

(2) The appropriate slip probability factor *D* must be selected from the *Guide* (Kulak et al., 1987) for design at the service-load level.

Prior to the 1994 edition of this Specification, μ for Class C surfaces was taken as 0.40. This value was reduced to 0.35 in the 1994 edition for better agreement with the available research (Kulak et al., 1987; pp. 78-82).

5.5. Tensile Fatigue

The tensile stress in the bolt that results from the cyclic application of externally applied service loads and the prying force, if any, but not the pretension, shall not exceed the stress in Table 5.2. The nominal diameter of the bolt shall be used in

Table 5.2. Maximum Tensile Stress for Fatigue Loading

Number of Cycles	Maximum Bolt Stress for Design at Service Loads[a], ksi	
	ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Not more than 20,000	44	54
From 20,000 to 500,000	40	49
More than 500,000	31	38

[a] Including the effects of *prying action*, if any, but excluding the pretension.

calculating the bolt stress. The connected parts shall be proportioned so that the calculated prying force does not exceed 30 percent of the externally applied load. *Joints* that are subject to tensile fatigue loading shall be specified as pretensioned in accordance with Section 4.2 or slip-critical in accordance with Section 4.3.

Commentary:

As described in the Commentary to Section 5.1, *high-strength bolts* in *pretensioned joints* that are nominally loaded in tension will experience little, if any, increase in axial stress under service loads. For this reason, pretensioned bolts are not adversely affected by repeated application of service-load tensile stress. However, care must be taken to ensure that the calculated prying force is a relatively small part of the total applied bolt tension (Kulak et al., 1987; p. 272). The provisions that cover bolt fatigue in tension are based upon research results where various single-bolt assemblies and *joints* with bolts in tension were subjected to repeated external loads that produced fatigue failure of the pretensioned fasteners. A limited range of prying effects was investigated in this research. As a matter of judgment, in this edition of the Specification the limit on prying forces as a percentage of the total externally applied tensile force has been reduced from 60 percent to 30 percent.

SECTION 6. USE OF WASHERS

6.1. Snug-Tightened Joints

Washers are not required in snug-tightened joints, except as required in Sections 6.1.1 and 6.1.2.

6.1.1. Sloping Surfaces: When the outer face of the *joint* has a slope that is greater than 1:20 with respect to a plane that is normal to the bolt axis, an ASTM F436 beveled washer shall be used to compensate for the lack of parallelism.

6.1.2. Slotted Hole: When a slotted hole occurs in an outer ply, an ASTM F436 washer or 5/16 in. thick common plate washer shall be used to cover the hole.

6.2 Pretensioned Joints and Slip-Critical Joints

Washers are not required in *pretensioned joints* and *slip-critical joints*, except as required in Sections 6.1.1, 6.1.2, 6.2.1, 6.2.2, 6.2.3, 6.2.4 and 6.2.5.

6.2.1. Specified Minimum Yield Strength of Connected Material Less Than 40 ksi: When ASTM A490 bolts are pretensioned in connected material of specified minimum yield strength less than 40 ksi, ASTM F436 washers shall be used under both the bolt head and nut, except that a washer is not needed under the head of an A490-strength round head twist-off bolt that meets the minimum bearing circle diameter requirements of ASTM F1852.

6.2.2. Calibrated Wrench Pretensioning: When the calibrated wrench pretensioning method is used, an ASTM F436 washer shall be used under the turned element.

6.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: When the twist-off-type tension-control bolt pretensioning method is used, an ASTM F436 washer shall be used under the nut as part of the *fastener assembly*.

6.2.4. Direct-Tension-Indicator Pretensioning: When the direct-tension-indicator pretensioning method is used, an ASTM F436 washer shall be used as follows:

(1) When the nut is turned and the direct tension indicator is located under the bolt head, an ASTM F436 washer shall be used under the nut;

(2) When the nut is turned and the direct tension indicator is located under the nut, an ASTM F436 washer shall be used between the nut and the direct tension indicator;

(3) When the bolt head is turned and the direct tension indicator is located under the nut, an ASTM F436 washer shall be used under the bolt head; and,

(4) When the bolt head is turned and the direct tension indicator is located under the bolt head, an ASTM F436 washer shall be used between the bolt head and the direct tension indicator.

6.2.5. Oversized or Slotted Hole: When an oversized or slotted hole occurs in an outer ply, the washer requirements shall be as given in Table 6.1. The washer used shall be of sufficient size to completely cover the hole.

Table 6.1. Washer Requirements for Bolted Joints with Oversized and Slotted Holes in the Outer Ply

ASTM Designation	Nominal Bolt Diameter, d_b, in.	Hole Type in Outer Ply		
		Oversized	Short-Slotted	Long-Slotted
A325 or F1852	½-1½	ASTM F436[a]		5/16-in.-thick plate washer or continuous bar[b, c]
A490	≤ 1			
	> 1	ASTM F436 with 5/16 in. thickness[b, d]		ASTM F436 washer with either a 3/8-in.-thick structural grade plate washer or continuous bar[b]

[a] This requirement shall not apply to heads of round head tension-control bolt assemblies that meet the requirements in Section 2.7 and provide a bearing circle diameter that meets the requirements of ASTM F1852.

[b] Multiple washers with a combined thickness of 5/16 in. or larger do not satisfy this requirement.

[c] The plate washer or bar shall be of structural-grade steel material, but need not be hardened.

[d] Alternatively, a 3/8-in.-thick plate washer and an ordinary thickness F436 washer may be used. The plate washer need not be hardened.

Commentary:
It is important that shop drawings and *connection* details clearly reflect the number and disposition of washers when they are required, especially the thick hardened washers or plate washers that are required for some slotted hole applications. The total thickness of washers in the *grip* affects the length of bolt that must be supplied and used.

The primary function of washers is to provide a hardened non-galling surface under the turned element, particularly for torque-based pretensioning methods such as the calibrated wrench pretensioning method and twist-off-type tension-control bolt pretensioning method. Circular flat washers that meet the requirements of ASTM F436 provide both a hardened non-galling surface and an increase in bearing area that is approximately 50 percent larger than that provided by a heavy-hex bolt head or nut. However, tests have shown that washers of the standard 5/32 in. thickness have a minor influence on the pressure distribution of the induced bolt pretension. Furthermore, they showed that a larger thickness is required when ASTM A490 bolts are used with material that has a minimum specified yield strength that is less than 40 ksi. This is necessary to mitigate the effects of local yielding of the material in the vicinity of the contact area of the head and nut. The requirement for standard thickness hardened washers, when such washers are specified, is waived for alternative design fasteners that incorporate a bearing surface under the head of the same diameter as the hardened washer.

Heat-treated washers not less than 5/16 in. thick are required to cover oversized and short-slotted holes in external plies, when ASTM A490 bolts of diameter larger than 1 in. are used, except per footnote d. This was found necessary to distribute the high clamping pressure so as to prevent collapse of the hole perimeter and enable the development of the desired clamping force. Preliminary investigation has shown that a similar but less severe deformation occurs when oversized or slotted holes are in the interior plies. The reduction in clamping force may be offset by "keying," which tends to increase the resistance to slip. These effects are accentuated in *joints* of thin plies. When long-slotted holes occur in an outer ply, 3/8 in. thick plate washers or continuous bars and one ASTM F436 washer are required in Table 6.1. This requirement can be satisfied with material of any structural grade. Alternatively, either of the following options can be used:

(1) The use of material with F_y greater than 40 ksi will eliminate the need to also provide ASTM F436 washers in accordance with the requirements in Section 6.2.1 for ASTM A490 bolts of any diameter.
(2) Material with F_y equal to or less than 40 ksi can be used with ASTM F436 washers in accordance with the requirements in Section 6.2.1

This specification previously required a washer under bolt heads with a bearing area smaller than that provided by an F436 washer. Tests indicate that the pretension achieved with a bolt having the minimum F1852 bearing circle diameter is the same as that of a bolt with the larger bearing circle diameter equal to the size of an F436 washer, provided that the hole size meets the RCSC Specification limitations. (Schnupp, 2003)

SECTION 7. PRE-INSTALLATION VERIFICATION

The requirements in this Section shall apply only as indicated in Sections 8.2 to verify that the *fastener assemblies* and pretensioned installation procedures perform as required prior to installation.

7.1. Tension Calibrator

A *tension calibrator* shall be used where bolts are to be installed in *pretensioned joints* and *slip-critical joints* to:

(1) Confirm the suitability of the complete *fastener assembly*, including lubrication, for pretensioned installation; and,
(2) Confirm the procedure and proper use by the bolting crew of the pretensioning method to be used.

The accuracy of the *tension calibrator* shall be confirmed through calibration at least annually.

Commentary:
A *tension calibrator* is a hydraulic device that indicates the pretension that is developed in a bolt that is installed in it. Such a device is an economical and valuable tool and it must be readily available whenever *high-strength bolts* are to be pretensioned. A bolt *tension calibrator* is essential for:

(1) The pre-installation verification of the suitability of the *fastener assembly*, including the lubrication that is applied by the *manufacturer* or specially applied, to develop the specified minimum pretension;
(2) Verifying the adequacy and proper use of the specified pretensioning method to be used;
(3) Determining the installation torque for the calibrated wrench pretensioning method; and,
(4) Determining an arbitration torque as specified in Section 10, if required to resolve dispute.

It is the only economically available tool for the described essential uses in the shop and field.

Hydraulic *tension calibrators* undergo a slight deformation during bolt pretensioning. Hence, when bolts are pretensioned according to Section 8.2.1, the nut rotation corresponding to a given pretension reading may be somewhat larger than it would be if the same bolt were pretensioned in a solid steel assembly. Stated differently, the reading of an hydraulic *tension calibrator* tends to underestimate the pretension that a given rotation of the turned element would induce in a bolt in a *pretensioned joint*.

7.2. Required Testing

A representative sample of not fewer than three complete *fastener assemblies* of each combination of diameter, length, grade and *lot* to be used in the work shall be checked at the site of installation in a *tension calibrator* to verify that the pretensioning method develops a pretension that is equal to or greater than 1.05 times that specified for installation and inspection in Table 8.1. Washers shall be used in the pre-installation verification assemblies as required in the work in accordance with the requirements in Section 6.2.

If the actual pretension developed in any of the *fastener assemblies* is less than 1.05 times that specified for installation and inspection in Table 8.1, the cause(s) shall be determined and resolved before the *fastener assemblies* are used in the work. Cleaning, lubrication and retesting of these *fastener assemblies*, except ASTM F1852 twist-off-type tension-control bolt assemblies, (see Section 2.2) are permitted, provided that all assemblies are treated in the same manner.

Commentary:

The fastener components listed in Section 1.3 are manufactured under separate ASTM specifications, each of which includes tolerances that are appropriate for the individual component covered. While these tolerances are intended to provide for a reasonable and workable fit between the components when used in an assembly, the cumulative effect of the individual tolerances permits a significant variation in the installation characteristics of the complete *fastener assembly*. It is the intent in this Specification that the responsibility rests with the *supplier* for proper performance of the *fastener assembly*, the components of which may have been produced by more than one *manufacturer*.

When pretensioned installation is required, it is essential that the effects of the accumulation of tolerances, surface condition and lubrication be taken into account. Hence, pre-installation verification testing of the complete *fastener assembly* is required as indicated in Section 8 to ensure that the *fastener assemblies* and installation method to be used in the work will provide a pretension that exceeds those specified in Table 8.1. It is not, however, intended simply to verify conformance with the individual ASTM specifications.

It is recognized in this Specification that a natural scatter is found in the results of the pre-installation verification testing that is required in Section 8. Furthermore, it is recognized that the pretensions developed in tests of a representative sample of the fastener components that will be installed in the work must be slightly higher to provide confidence that the majority of *fastener assemblies* will achieve the minimum required pretension as given in Table 8.1. Accordingly, the minimum pretension to be used in pre-installation verification is 1.05 times that required for installation and inspection.

Pre-installation verification testing of as-received bolts and nuts is also a requirement in this Specification because of instances of under-strength and counterfeit bolts and nuts. Pre-installation verification testing provides a practical means for ensuring that non-conforming *fastener assemblies* are not incorporated into the work. Experience on many projects has shown that bolts and/or nuts not meeting the requirements of the applicable ASTM Specification would have been identified prior to installation if they had been tested as an assembly in a *tension calibrator*. The expense of replacing bolts installed in the structure when the non-conforming bolts were discovered at a later date would have been avoided.

Additionally, pre-installation verification testing clarifies for the bolting crew and the *inspector* the proper implementation of the selected pretensioning method and the adequacy of the installation equipment. It will also identify potential sources of problems, such as the need for lubrication to prevent failure of bolts by combined high torque with tension, under-strength assemblies resulting from excessive over-tapping of hot-dip galvanized nuts or other failures to meet strength or geometry requirements of applicable ASTM specifications.

The pre-installation verification requirements in this Section presume that *fastener assemblies* so verified will be pretensioned before the condition of the *fastener assemblies*, the equipment and the steelwork have changed significantly. Research by Kulak and Undershute (1998) on twist-off-type tension-control bolt assemblies from various *manufacturers* showed that installed pretensions could be a function of the time and environmental conditions of storage and exposure. The reduced performance of these bolts was caused by a deterioration of the lubricity of the assemblies. Furthermore, all bolt pre-tensioning that is achieved through rotation of the nut (or the head) is affected by the presence of torque, the excess of which has been demonstrated to adversely affect the development of the desired pretension. Thus, it is required that the condition of the *fastener assemblies* must be replicated in pre-installation verification. When time of exposure between the placement of *fastener assemblies* in the field work and the subsequent pretensioning of those *fastener assemblies* is of concern, pre-installation verification can be performed on *fastener assemblies* removed from the work or on extra *fastener assemblies* that, at the time of placement, were set aside to experience the same degree of exposure.

SECTION 8. INSTALLATION

Prior to installation, the fastener components shall be stored in accordance with Section 2.2. For *joints* that are designated in the contract documents as *snug-tightened joints*, the bolts shall be installed in accordance with Section 8.1. For *joints* that are designated in the contract documents as pretensioned or slip-critical, the bolts shall be installed in accordance with Section 8.2.

8.1. Snug-Tightened Joints

All bolt holes shall be aligned to permit insertion of the bolts without undue damage to the threads. Bolts shall be placed in all holes with washers positioned as required in Section 6.1 and nuts threaded to complete the assembly. Compacting the *joint* to the snug-tight condition shall progress systematically from the most rigid part of the *joint*. The snug-tightened condition is the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the connected plies into *firm contact*.

Commentary:

As discussed in the Commentary to Section 4, the bolted *joints* in most shear *connections* and in many tension *connections* can be specified as *snug-tightened joints*. The snug-tightened condition is the tightness that is attained with a few impacts of an impact wrench or the full effort of an ironworker using an ordinary spud wrench to bring the plies into *firm contact*. More than one cycle through the bolt pattern may be required to achieve the *snug-tightened joint*.

The actual pretensions that result in individual fasteners in *snug-tightened joints* will vary from *joint* to *joint* depending upon the thickness, flatness, and degree of parallelism of the connected plies as well as the effort applied. In most *joints*, plies of *joints* involving material of ordinary thickness and flatness can be drawn into complete contact at relatively low levels of pretension. However, in some *joints* in thick material, or in material with large burrs, it may not be possible to reach continuous contact throughout the *faying surface* area as is commonly achieved in *joints* of thinner plates. This is generally not detrimental to the performance of the *joint*.

As used in Section 8.1, the term “undue damage” is intended to mean damage that would be sufficient to render the product unfit for its intended use.

8.2. Pretensioned Joints

One of the pretensioning methods in Sections 8.2.1 through 8.2.4 shall be used, except when alternative-design fasteners that meet the requirements of Section 2.8 or alternative washer-type indicating devices that meet the requirements of Section 2.6.2 are used, in which case, installation instructions provided by the *manufacturer* and approved by the *Engineer of Record* shall be followed. When it is impractical to turn the nut, pretensioning by turning the bolt head is permitted while rotation of the nut is prevented, provided that the washer requirements in Section 6.2 are met. A pretension that is equal to or greater than the value in Table 8.1 shall be provided. The pre-installation verification procedures specified in Section 7 shall be performed using *fastener assemblies* that are representative of the condition of those that will be pretensioned in the work.

Table 8.1. Minimum Bolt Pretension for Pretensioned and Slip-Critical Joints

Nominal Bolt Diameter d_b, in.	Specified Minimum Bolt Pretension T_m, kips[a]	
	ASTM A325 and F1852 Bolts	ASTM A490 Bolts
1/2	12	15
5/8	19	24
3/4	28	35
7/8	39	49
1	51	64
1 1/8	56	80
1 1/4	71	102
1 3/8	85	121
1 1/2	103	148

[a] Equal to 70 percent of the specified minimum tensile strength of bolts as specified in ASTM Specifications for tests of full-size ASTM A325 and A490 bolts with UNC threads loaded in axial tension, rounded to the nearest kip.

Commentary:

The minimum pretension for ASTM A325 and A490 bolts is equal to 70 percent of the specified minimum tensile strength. As tabulated in Table 8.1, the values have been rounded to the nearest kip.

Four pretensioning methods are provided without preference in this Specification. Each method may be relied upon to provide satisfactory results when conscientiously implemented with the specified *fastener assembly* components in good condition. However, it must be recognized that misuse or abuse is possible with any method. With all methods, it is important to first install bolts in all holes of the *joint* and to compact the *joint* until the connected plies are in *firm contact*. Only after completion of this operation can the *joint* be reliably pretensioned. Both the initial phase of compacting the *joint* and the subsequent phase of pretensioning should begin at the most rigidly fixed or stiffest point.

In some *joints* in thick material, it may not be possible to reach continuous contact throughout the *faying surface* area, as is commonly achieved in *joints* of thinner plates. This is not detrimental to the performance of the *joint*. If the specified pretension is present in all bolts of the completed *joint*, the clamping force, which is equal to the total of the pretensions in all bolts, will be transferred at the locations that are in contact and the *joint* will be fully effective in resisting slip through friction.

If individual bolts are pretensioned in a single continuous operation in a *joint* that has not first been properly compacted or fitted up, the pretension in the bolts that are pretensioned first may be relaxed or removed by the pretensioning of adjacent bolts. The resulting reduction in total clamping force will reduce the slip resistance.

In the case of hot-dip galvanized coatings, especially if the *joint* consists of many plies of thickly coated material, relaxation of bolt pretension may be significant and re-pretensioning of the bolts may be required subsequent to the initial pretensioning. Munse (1967) showed that a loss of pretension of approximately 6.5 percent occurred for galvanized plates and bolts due to relaxation as compared with 2.5 percent for uncoated *joints*. This loss of bolt pretension occurred in five days; loss recorded thereafter was negligible. Either this loss can be allowed for in design or pretension may be brought back to the prescribed level by re-pretensioning the bolts after an initial period of "settling-in".

As stated in the *Guide*, Kulak et al (1987; p. 61), "…it seems reasonable to expect an increase in bolt force relaxation as the *grip* length is decreased. Similarly, increasing the number of plies for a constant *grip* length might also lead to an increase in bolt relaxation."

8.2.1. Turn-of-Nut Pretensioning: All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. Subsequently, the nut or head rotation specified in Table 8.2 shall be applied to all *fastener assemblies* in the *joint*, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The part not turned by the wrench shall be prevented from rotating during this operation.

Table 8.2. Nut Rotation from Snug-Tight Condition for Turn-of-Nut Pretensioning [a,b]

Bolt Length[c]	Disposition of Outer Face of Bolted Parts		
	Both faces normal to bolt axis	One face normal to bolt axis, other sloped not more than 1:20[d]	Both faces sloped not more than 1:20 from normal to bolt axis[d]
Not more than $4d_b$	⅓ turn	½ turn	⅔ turn
More than $4d_b$ but not more than $8d_b$	½ turn	⅔ turn	⅚ turn
More than $8d_b$ but not more than $12d_b$	⅔ turn	⅚ turn	1 turn

[a] Nut rotation is relative to bolt regardless of the element (nut or bolt) being turned. For required nut rotations of ½ turn and less, the tolerance is plus or minus 30 degrees; for required nut rotations of ⅔ turn and more, the tolerance is plus or minus 45 degrees.

[b] Applicable only to joints in which all material within the grip is steel.

[c] When the bolt length exceeds $12d_b$, the required nut rotation shall be determined by actual testing in a suitable tension calibrator that simulates the conditions of solidly fitting steel.

[d] Beveled washer not used.

Commentary:
The turn-of-nut pretensioning method results in more uniform bolt pretensions than is generally provided with torque-controlled pretensioning methods. Strain-control that reaches the inelastic region of bolt behavior is inherently more reliable than a method that is dependent upon torque control. However, proper implementation is dependent upon ensuring that the *joint* is properly compacted prior to application of the required partial turn and that the bolt head (or nut) is securely held when the nut (or bolt head) is being turned.

Match-marking of the nut and protruding end of the bolt after snug-tightening can be helpful in the subsequent installation process, and is certainly an aid to inspection.

As indicated in Table 8.2, there is no available research that establishes the required nut rotation for bolt lengths exceeding $12d_b$. The required turn for such bolts can be established on a case-by-case basis using a *tension calibrator*.

8.2.2. Calibrated Wrench Pretensioning: The pre-installation verification procedures specified in Section 7 shall be performed daily for the calibration of the installation wrench. Torque values determined from tables or from equations that claim to relate torque to pretension without verification shall not be used.

All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. Subsequently, the installation torque determined in the pre-installation verification of the *fastener assembly* (Section 7) shall be applied to all bolts in the *joint*, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The part not turned by the wrench shall be prevented from rotating during this operation. Application of the installation torque need not produce a relative rotation between the bolt and nut that is greater than the rotation specified in Table 8.2.

Commentary:
The scatter in installed pretension can be significant when torque-controlled methods of installation are used. The variables that affect the relationship between torque and pretension include:

(1) The finish and tolerance on the bolt and nut threads;
(2) The uniformity, degree and condition of lubrication;
(3) The shop or job-site conditions that contribute to dust and dirt or corrosion on the threads;
(4) The friction that exists to a varying degree between the turned element (the nut face or bearing area of the bolt head) and the supporting surface;
(5) The variability of the air supply parameters on impact wrenches that results from the length of air lines or number of wrenches operating from the same source;
(6) The condition, lubrication and power supply for the torque wrench, which may change within a work shift; and,
(7) The repeatability of the performance of any wrench that senses or responds to the level of the applied torque.

In the first edition of this Specification, which was published in 1951, a table of torque-to-pretension relationships for bolts of various diameters was included. It was soon demonstrated in research that a variation in the torque-to-pretension of as high as ±40 percent must be anticipated unless the relationship is established individually for each bolt *lot*, diameter, and fastener condition. Hence, in the 1954 edition of this Specification, recognition of relationships between torque and pretension in the form of tabulated values or equations was withdrawn. Recognition of the calibrated wrench pretensioning method was retained however until 1980, but with the requirement that the torque required for installation be determined specifically for the bolts being installed on a daily basis. Recognition of the method was withdrawn in 1980 because of the continuing controversy that resulted from the failure of users to adhere to the requirements for the valid use of the method during both installation and inspection.

In the 1985 edition of this Specification, the calibrated wrench pretensioning method was reinstated, but with more emphasis on detailed requirements that must be carefully followed. For calibrated wrench pretensioning, wrenches must be calibrated:

(1) Daily;
(2) When the *lot* of any component of the *fastener assembly* is changed;
(3) When the *lot* of any component of the *fastener assembly* is relubricated;
(4) When significant differences are noted in the surface condition of the bolt threads, nuts or washers; or,
(5) When any major component of the wrench including lubrication, hose and air supply are altered.

It is also important that:

(1) Fastener components be protected from dirt and moisture at the shop or jobsite as required in Section 2;
(2) Washers be used as specified in Section 6; and
(3) The time between removal from *protected storage* and wrench calibration and final pretensioning be minimal.

8.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: Twist-off-type tension-control bolt assemblies that meet the requirements of ASTM F1852 shall be used.

All *fastener assemblies* shall be installed in accordance with the requirements in Section 8.1 without severing the splined end and with washers positioned as required in Section 6.2. If a splined end is severed during this operation, the *fastener assembly* shall be removed and replaced. Subsequently, all bolts in the *joint* shall be pretensioned with the twist-off-type tension-control bolt installation wrench, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts.

Commentary:
ASTM F1852 twist-off-type tension-control bolt assemblies have a splined end that extends beyond the threaded portion of the bolt. During installation, this

splined end is gripped by a specially designed wrench chuck and provides a means for turning the nut relative to the bolt. This product is, in fact, based upon a torque-controlled installation method to which the *fastener assembly* variables affecting torque that were discussed in the Commentary to Section 8.2.2 apply, except for wrench calibration, because torque is controlled within the *fastener assembly*.

Twist-off-type tension-control bolt assemblies must be used in the as-delivered, clean, lubricated condition as specified in Section 2. Adherence to the requirements in this Specification, especially those for storage, cleanliness and verification, is necessary for their proper use.

8.2.4. Direct-Tension-Indicator Pretensioning: Direct tension indicators that meet the requirements of ASTM F959 shall be used. The pre-installation verification procedures specified in Section 7 shall demonstrate that, when the pretension in the bolt reaches 1.05 times that specified for installation and inspection in Table 8.1, the gap is not less than the job inspection gap in accordance with ASTM F959.

All bolts shall be installed in accordance with the requirements in Section 8.1, with washers positioned as required in Section 6.2. The installer shall verify that the direct-tension-indicator protrusions have not been compressed to a gap that is less than the job inspection gap during this operation, and if this has occurred, the direct tension indicator shall be removed and replaced. Subsequently, all bolts in the *joint* shall be pretensioned, progressing systematically from the most rigid part of the *joint* in a manner that will minimize relaxation of previously pretensioned bolts. The installer shall verify that the direct tension indicator protrusions have been compressed to a gap that is less than the job inspection gap.

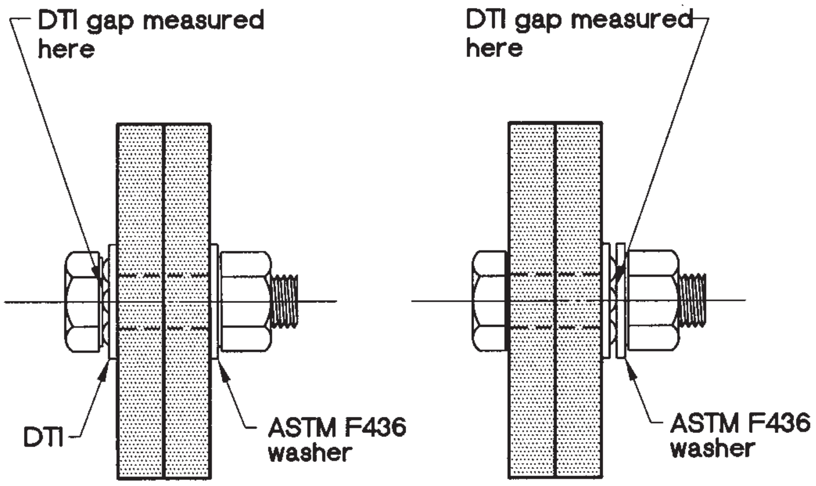
Commentary:

ASTM F959 direct tension indicators are recognized in this Specification as a bolt-tension-indicating device. Direct tension indicators are hardened, washer-shaped devices incorporating small arch-like protrusions on the bearing surface that are designed to deform in a controlled manner when subjected to compressive load.

During installation, care must be taken to ensure that the direct-tension-indicator arches are oriented to bear against the hardened bearing surface of the bolt head or nut or against a hardened flat washer if used under turned element whether that turned element is the nut or the bolt. Proper use and orientation is illustrated in Figure C-8.1.

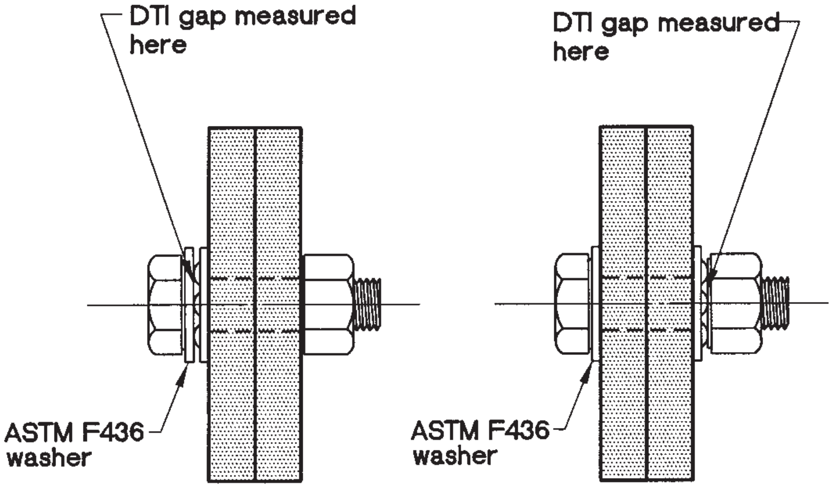
In some cases, more than a single cycle of systematic partial pretensioning may be required to deform the direct-tension-indicator protrusions to the gap that is specified by the *manufacturer*. If the gaps fail to close or when the washer *lot* is changed, another verification procedure using the *tension calibrator* must be performed.

Provided the connected plies are in *firm contact*, partial compression of the direct tension indicator protrusions is commonly taken as an indication that the snug-tight condition has been achieved.



a) DTI under bolt head, nut turned

b) DTI under nut, nut turned



c) DTI under bolt head, bolt head turned

d) DTI under nut, bolt head turned

Note: See Section 6, for general requirements for the use of washers.

Figure. C-8.1. Proper use and orientation of ASTM F959 direct-tension indicator.

SECTION 9. INSPECTION

When inspection is required in the contract documents, the *inspector* shall ensure while the work is in progress that the requirements in this Specification are met. When inspection is not required in the contract documents, the *contractor* shall ensure while the work is in progress that the requirements in this Specification are met.

For *joints* that are designated in the contract documents as *snug-tightened joints*, the inspection shall be in accordance with Section 9.1. For *joints* that are designated in the contract documents as pretensioned, the inspection shall be in accordance with Section 9.2. For *joints* that are designated in the contract documents as slip-critical, the inspection shall be in accordance with Section 9.3.

9.1. Snug-Tightened Joints

Prior to the *start of work*, it shall be ensured that all fastener components to be used in the work meet the requirements in Section 2. Subsequently, it shall be ensured that all connected plies meet the requirements in Section 3.1 and all bolt holes meet the requirements in Sections 3.3 and 3.4. After the *connections* have been assembled, it shall be visually ensured that the plies of the connected elements have been brought into *firm contact* and that washers have been used as required in Section 6. No further evidence of conformity is required for *snug-tightened joints*. The magnitude of the clamping force that exists in a *snug-tightened joint* is not a consideration.

Commentary:

Inspection requirements for *snug-tightened joints* consist of verification that the proper fastener components were used, the connected elements were fabricated properly, and the bolted *joint* was drawn into firm contact. Because pretension is not required for the proper performance of a *snug-tightened joint*, the installed bolts should not be inspected to determine the actual installed pretension. Likewise, the arbitration procedures described in Section 10 are not appropriate.

9.2. Pretensioned Joints

For *pretensioned joints*, the following inspection shall be performed in addition to that required in Section 9.1:

(1) When the turn-of-nut pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.1.

(2) When the calibrated wrench pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.2.

(3) When the twist-off-type tension-control bolt pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.3.

(4) When the direct-tension-indicator pretensioning method is used for installation, the inspection shall be in accordance with Section 9.2.4.

(5) When alternative-design fasteners that meet the requirements of Section 2.8 or alternative washer-type indicating devices that meet the requirements of Section 2.6.2 are used, the inspection shall be in accordance with inspection instructions provided by the manufacturer and approved by the Engineer of Record.

Commentary:
When *joints* are designated as pretensioned, they are not subject to the same faying-surface-treatment inspection requirements as is specified for *slip-critical joints* in Section 9.3.

9.2.1. Turn-of-Nut Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.1. Subsequently, it shall be ensured by *routine observation* that the bolting crew properly rotates the turned element relative to the unturned element by the amount specified in Table 8.2. Alternatively, when *fastener assemblies* are match-marked after the initial fit-up of the *joint* but prior to pretensioning, visual inspection after pretensioning is permitted in lieu of routine observation. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
Match-marking of the assembly during installation as discussed in the Commentary to Section 8.2.1 improves the ability to inspect bolts that have been pretensioned with the turn-of-nut pretensioning method. The sides of nuts and bolt heads that have been impacted sufficiently to induce the Table 8.1 minimum pretension will appear slightly peened.

The turn-of-nut pretensioning method, when properly applied and verified during the construction, provides more reliable installed pretensions than after-the-fact *inspection* testing. Therefore, proper inspection of the method is for the inspector to observe the required pre-installation verification testing of the *fastener assemblies* and the method to be used, followed by monitoring of the work in progress to ensure that the method is routinely and properly applied, or visual inspection of match-marked assemblies.

Some problems with the turn-of-nut pretensioning method have been encountered with hot-dip galvanized bolts. In some cases, the problems have been attributed to an especially effective lubricant applied by the *manufacturer* to ensure that bolts and nuts from stock will meet the ASTM Specification requirements for minimum turns testing of galvanized fasteners. Job-site testing in the *tension calibrator* demonstrated that the lubricant reduced the coefficient of friction between the bolt and nut to the degree that “the full effort of an ironworker using an ordinary spud wrench” to snug-tighten the *joint* actually induced the full required pretension. Also, because the nuts could be removed with an ordinary spud wrench, they were erroneously judged by the *inspector* to be improperly pretensioned. Excessively lubricated *high-strength bolts* may require significantly less torque to induce the specified pretension. The required pre-installation verification will reveal this potential problem.

Conversely, the absence of lubrication or lack of proper over-tapping can cause seizing of the nut and bolt threads, which will result in a twist failure of the bolt at less than the specified pretension. For such situations, the use of a *tension calibrator* to check the bolt assemblies to be installed will be helpful in establishing the need for lubrication.

9.2.2. Calibrated Wrench Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.2. Subsequently, it shall be ensured by *routine observation* that the bolting crew properly applies the calibrated wrench to the turned element. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
For proper inspection of the method, it is necessary for the *inspector* to observe the required pre-installation verification testing of the *fastener assemblies* and the method to be used, followed by monitoring of the work in progress to ensure that the method is routinely and properly applied within the limits on time between removal from *protected storage* and final pretensioning.

9.2.3. Twist-Off-Type Tension-Control Bolt Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.3. Subsequently, it shall be ensured by *routine observation* that the splined ends are properly severed during installation by the bolting crew. No further evidence of conformity is required. A pretension that is greater than the value specified in Table 8.1 shall not be cause for rejection.

Commentary:
The sheared-off splined end of an installed ASTM F1852 twist-off-type tension-control bolt assembly merely signifies that at some time the bolt was subjected to a torque that was adequate to cause the shearing. If in fact all fasteners are individually pretensioned in a single continuous operation without first properly snug-tightening all fasteners, they may give a misleading indication that the bolts have been properly pretensioned. Therefore it is necessary that the *inspector* observe the required pre-installation verification testing of the *fastener assemblies*, and the ability to apply partial tension prior to twist-off is demonstrated. This is followed by monitoring of the work in progress to ensure that the method is routinely and properly applied within the limits on time between removal from *protected storage* and final twist-off of the splined end.

9.2.4. Direct-Tension-Indicator Pretensioning: The *inspector* shall observe the pre-installation verification testing required in Section 8.2.4. Subsequently, but prior to pretensioning, it shall be ensured by *routine observation* that the appropriate feeler gage is accepted in at least half of the spaces between the protrusions of the direct tension indicator and that the protrusions are properly oriented away from the work. If the appropriate feeler gage is accepted in fewer than half of the spaces, the direct tension indicator shall be removed and replaced. After pretensioning, it shall be ensured by *routine observation* that the appropriate feeler gage is refused entry into at least half of the spaces between the protrusions. No further evidence of conformity is required. A pretension that is greater than that specified in Table 8.1 shall not be cause for rejection.

Commentary:

When the *joint* is initially snug tightened, the direct tension indicator arch-like protrusions will generally compress partially. Whenever the snug-tightening operation causes one-half or more of the gaps between these arch-like protrusions to close to 0.015 in. or less (0.005 in. or less for coated direct tension indicators), the direct tension indicator should be replaced. Only after this initial operation should the bolts be pretensioned in a systematic manner. If the bolts are installed and pretensioned in a single continuous operation, direct tension indicators may give the *inspector* a misleading indication that the bolts have been properly pretensioned. Therefore, it is necessary that the *inspector* observe the required pre-installation verification testing of the *fastener assemblies* with the direct-tension indicators properly located and the method to be used. Following this operation, the *inspector* should monitor the work in progress to ensure that the method is routinely and properly applied.

9.3. Slip-Critical Joints

Prior to assembly, it shall be visually verified that the *faying surfaces* of *slip-critical joints* meet the requirements in Section 3.2.2. Subsequently, the inspection required in Section 9.2 shall be performed.

Commentary:

When *joints* are specified as slip-critical, it is necessary to verify that the *faying surface* condition meets the requirements as specified in the contract documents prior to assembly of the *joint* and that the bolts are properly pretensioned after they have been installed. Accordingly, the inspection requirements for *slip-critical joints* are identical to those specified in Section 9.2, with additional *faying surface* condition inspection requirements.

SECTION 10. ARBITRATION

When it is suspected after inspection in accordance with Section 9.2 or Section 9.3 that bolts in pretensioned or *slip-critical joints* do not have the proper pretension, the following arbitration procedure is permitted. If verification of bolt pretension is required after the passage of a period of time and exposure of the completed *joints*, an alternative arbitration procedure that is appropriate to the specific situation shall be used.

(1) A representative sample of five bolt and nut assemblies of each combination of diameter, length, grade and *lot* in question shall be installed in a *tension calibrator*. The material under the turned element shall be the same as in the actual installation; that is, structural steel or hardened washer. The bolt shall be partially pretensioned to approximately 15 percent of the pretension specified in Table 8.1. Subsequently, the bolt shall be pretensioned to the minimum value specified in Table 8.1.

(2) A manual torque wrench that indicates torque by means of a dial, or one that may be adjusted to give an indication that a defined torque has been reached, shall be applied to the pretensioned bolt. The torque that is necessary to rotate the nut or bolt head five degrees (approximately 1 in. at 12-in. radius) relative to its mating component in the tightening direction shall be determined. The arbitration torque shall be determined by rejecting the high and low values and averaging the remaining three.

(3) Bolts represented by the above sample shall be tested by applying, in the tightening direction, the arbitration torque to 10 percent of the bolts, but no fewer than two bolts, selected at random in each *joint* in question. If no nut or bolt head is turned relative to its mating component by application of the arbitration torque, the *joint* shall be accepted as properly pretensioned.

If any nut or bolt is turned relative to its mating component by an attempted application of the arbitration torque, all bolts in the *joint* shall be tested. Those bolts whose nut or head is turned relative to its mating component by application of the arbitration torque shall be re-pretensioned by the Fabricator or Erector and reinspected. Alternatively, the Fabricator or Erector, at their option, is permitted to re-pretension all of the bolts in the *joint* and subsequently resubmit the *joint* for inspection.

Commentary:
When bolt pretension is arbitrated using torque wrenches after pretensioning, such arbitration is subject to all of the uncertainties of torque-controlled calibrated wrench installation that are discussed in the Commentary to Section 8.2.2. Additionally, the reliability of after-the-fact torque wrench arbitration is reduced by the absence of many of the controls that are necessary to minimize the variability of the torque-to-pretension relationship, such as:

(1) The use of hardened washers[3];
(2) Careful attention to lubrication; and,
(3) The uncertainty of the effect of passage of time and exposure in the installed condition.

Furthermore, in many cases such arbitration may have to be based upon an arbitration torque that is determined either using bolts that can only be assumed to be representative of the bolts used in the actual job or using bolts that are removed from completed *joints*. Ultimately, such arbitration may wrongly reject bolts that were subjected to a properly implemented installation procedure. The arbitration procedure contained in this Specification is provided, in spite of its limitations, as the most feasible available at this time.

Arbitration using an ultrasonic extensometer or a mechanical one capable of measuring changes in bolt length can be performed on a sample of bolts that is representative of those that have been installed in the work. Several *manufacturers* produce equipment specifically for this application. The use of appropriate techniques, which includes calibration, can produce a very accurate measurement of the actual pretension. The method involves measurement of the change in bolt length during the release of the nut, combined with either a load calibration of the removed *fastener assembly* or a theoretical calculation of the force corresponding to the measured elastic release or "stretch". Reinstallation of the released bolt or installation of a replacement bolt is required.

The required release suggests that the direct use of extensometers as an inspection tool be used in only the most critical cases. The problem of reinstallation may require bolt replacement unless torque can be applied slowly using a manual or hydraulic wrench, which will permit the restoration of the original elongation.

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[3] For example, because the reliability of the turn-of-nut pretensioning method is not dependent upon the presence or absence of washers under the turned element, washers are not generally required, except for other reasons as indicated in Section 6. Thus, in the absence of washers,after-the-fact, torque-based arbitration is particularly unreliable when the turn-of-nut pretensioning method has been used for installation.

APPENDIX A. TESTING METHOD TO DETERMINE THE SLIP COEFFICIENT FOR COATINGS USED IN BOLTED JOINTS

SECTION A1. GENERAL PROVISIONS

A1.1. Purpose and Scope

The purpose of this testing procedure is to determine the *mean slip coefficient* of a coating for use in the design of *slip-critical joints*. Adherence to this testing method provides that the creep deformation of the coating due to both the clamping force of the bolt and the service-load *joint* shear are such that the coating will provide satisfactory performance under sustained loading.

Commentary:

The Research Council on Structural Connections on June 14, 1984, first approved the testing method developed by Yura and Frank (1985). It has since been revised to incorporate changes resulting from the intervening years of experience with the testing method, and is now included as an appendix to this Specification.

The slip coefficient under short-term static loading has been found to be independent of the magnitude of the clamping force, variations in coating thickness and bolt hole diameter.

The proposed test methods are designed to provide the necessary information to evaluate the suitability of a coating for *slip-critical joints* and to determine the *mean slip coefficient* to be used in the design of the *joints*. The initial testing of the compression specimens provides a measure of the scatter of the slip coefficient.

The creep tests are designed to measure the creep behavior of the coating under the service loads, determined by the slip coefficient of the coating based upon the compression test results. The slip test conducted at the conclusion of the creep test is to ensure that the loss of clamping force in the bolt does not reduce the slip load below that associated with the design slip coefficient. ASTM A490 bolts are specified, since the loss of clamping force is larger for these bolts than that for ASTM A325 bolts. Qualification of the coating for use in a structure at an average thickness of 2 mils less than that to be used for the test specimen is to ensure that a casual buildup of the coating due to overspray and other causes does not jeopardize the coating's performance.

A1.2. Definition of Essential Variables

Essential variables are those that, if changed, will require retesting of the coating to determine its *mean slip coefficient*. The essential variables and the relationship of these variables to the limitations of application of the coating for structural *joints* are given below. The slip coefficient testing shall be repeated if there is any change in these essential variables.

A1.2.1. Time Interval: The time interval between application of the coating and the time of testing is an essential variable. The time interval must be recorded in hours and any special curing procedures detailed. Curing according to published *manufacturer's* recommendations would not be considered a special curing procedure. The coatings are qualified for use in structural *connections* that are assembled after coating for a time equal to or greater than the interval used in the test specimens. Special curing conditions used in the test specimens will also apply to the use of the coating in the structural *connections*.

A1.2.2. Coating Thickness: The coating thickness is an essential variable. The maximum average coating thickness, as per SSPC PA2 (SSPC 1993; SSPC 1991), allowed on the faying surfaces is 2 mils less than the average thickness, rounded to the nearest whole mil, of the coating that is used on the test specimens.

A1.2.3. Coating Composition and Method of Manufacture: The composition of the coating, including the thinners used, and its method of manufacture are essential variables.

A1.3. Retesting

A coating that fails to meet the creep or the post-creep slip test requirements in Section A4 may be retested in accordance with methods in Section A4 at a lower slip coefficient without repeating the static short-term tests specified in Section A3. Essential variables shall remain unchanged in the retest.

SECTION A2. TEST PLATES AND COATING OF THE SPECIMENS

A2.1. Test Plates

The test specimen plates for the short-term static tests are shown in Figure A1. The plates are 4 in. × 4 in. × ⅝ in. thick, with a 1 in. diameter hole drilled 1½ in. ± 1⁄16 in. from one edge. The test specimen plates for the creep tests are shown in Figure A2. The plates are 4 in. × 7 in. × ⅝ in. thick with two 1 in. diameter holes drilled 1½ in. ± 1⁄16 in. from each end. The edges of the plates may be milled, as-rolled or saw-cut; thermally cut edges are not permitted. The plates shall be flat enough to ensure that they will be in reasonably full contact over the *faying surface*. All burrs, lips or rough edges shall be removed. The arrangement of the specimen plates for the testing is shown in Figure A2. The plates shall be fabricated from a steel with a specified minimum yield strength that is between 36 and 50 ksi.

If specimens with more than one bolt are desired, the contact surface per bolt shall be 4 in. × 3 in. as shown for the single-bolt specimen in Figure A1.

Commentary:

The use of 1-in.-diameter bolt holes in the specimens is to ensure that adequate clearance is available for slip. Fabrication tolerances, coating buildup on the holes, and assembly tolerances tend to reduce the apparent clearances.

A2.2. Specimen Coating

Coatings are to be applied to the specimens in a manner that is consistent with that to be used in the actual intended structural application. The method of applying the coating and the surface preparation shall be given in the test report. The specimens are to be coated to an average thickness that is 2 mils greater than the maximum thickness to be used in the structure on both of the plate surfaces (the faying and outer surfaces). The thickness of the total coating and the primer, if used, shall be measured on the contact surface of the specimens. The thickness shall be measured in accordance with SSPC-PA2 (SSPC, 1993; SSPC, 1991). Two spot readings (six gage readings) shall be made for each contact surface. The overall average thickness from the three plates comprising a specimen is the average thickness for the specimen. This value shall be reported for each specimen. The average coating thickness of the creep specimens shall be calculated and reported.

The time between application of the coating and specimen assembly shall be the same for all specimens within ±4 hours. The average time shall be calculated and reported.

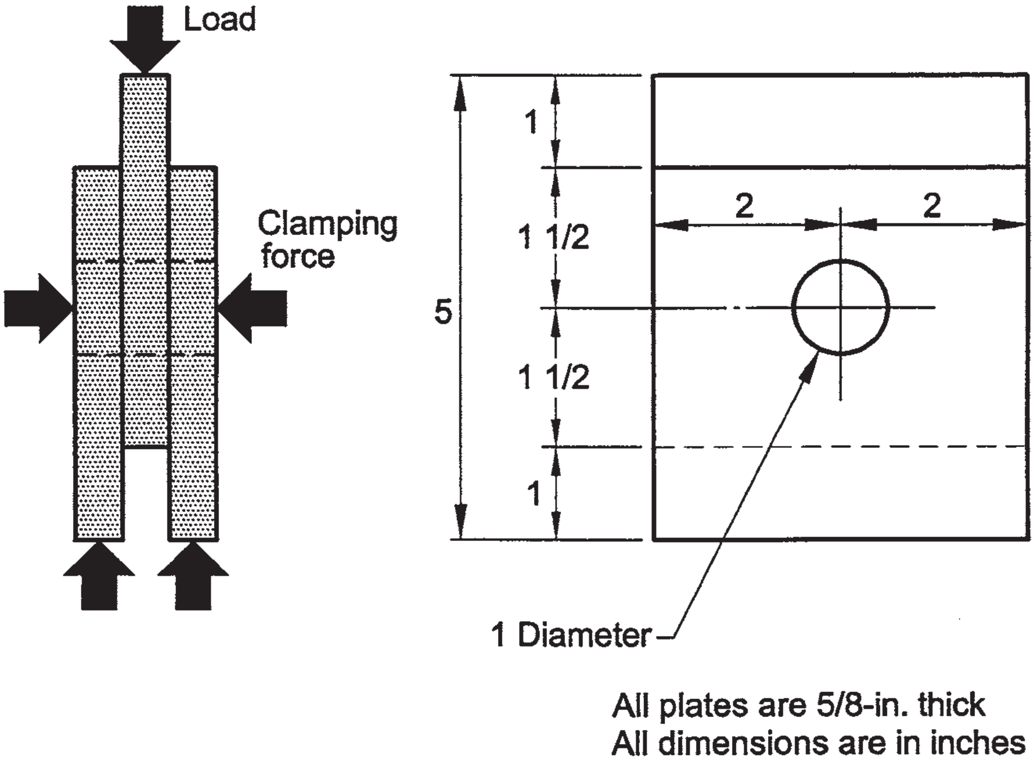
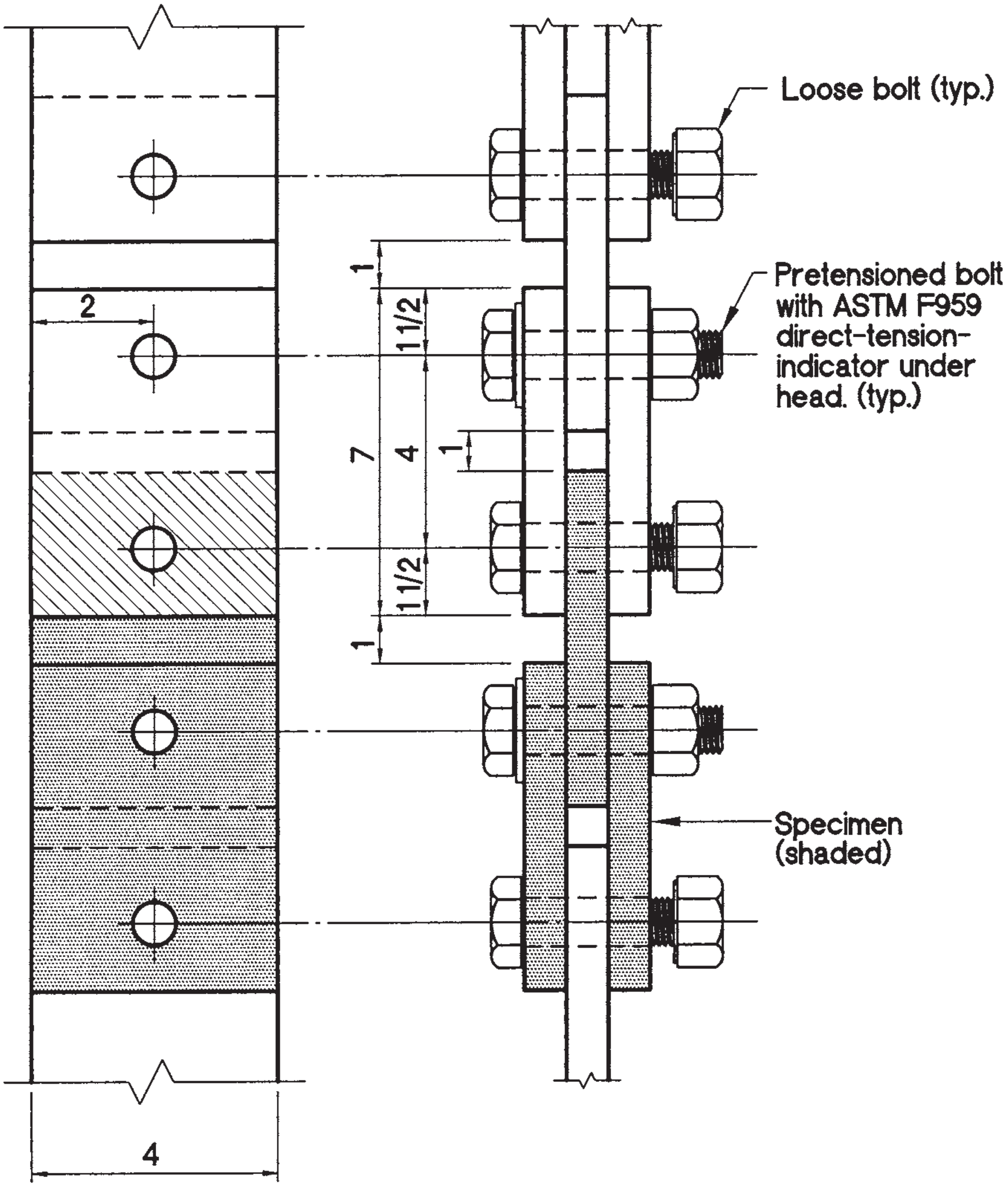


Figure A-1. Compression slip test specimen.



All dimensions are typical
All plates are 5/8-in. thick
All dimensions are in inches

Figure A-2. Creep test specimen assembly.

SECTION A3. SLIP TESTS

The methods and procedures described herein are used to experimentally determine the *mean slip coefficient* under short-term static loading for *high-strength bolted joints*. The *mean slip coefficient* shall be determined by testing one set of five specimens.

Commentary:
The slip load measured in this setup yields the slip coefficient directly since the clamping force is controlled and measured directly. The resulting slip coefficient has been found to correlate with both tension and compression tests of bolted specimens. However, tests of bolted specimens revealed that the clamping force may not be constant but decreases with time due to the compressive creep of the coating on the *faying surfaces* and under the nut and bolt head. The reduction in clamping force can be considerable for *joints* with high clamping force and thick coatings (as much as a 20 percent loss). This reduction in clamping force causes a corresponding reduction in the slip load. The resulting reduction in slip load must be considered in the procedure used to determine the design allowable slip loads for the coating.

The loss in clamping force is a characteristic of the coating. Consequently, it cannot be accounted for by an increase in the factor of safety or a reduction in the clamping force used for design without unduly penalizing coatings that do not exhibit this behavior.

A3.1. Compression Test Setup

The test setup shown in Figure A3 has two major loading components, one to apply a clamping force to the specimen plates and another to apply a compressive load to the specimen so that the load is transferred across the *faying surfaces* by friction.

A3.1.1. Clamping Force System: The clamping force system consists of a ⅞ in. diameter threaded rod that passes through the specimen and a centerhole compression ram. An ASTM A563 grade DH nut is used at both ends of the rod and a hardened washer is used at each side of the test specimen. Between the ram and the specimen is a specially modified ⅞ in. diameter ASTM A563 grade DH nut in which the threads have been drilled out so that it will slide with little resistance along the rod. When oil is pumped into the centerhole ram, the piston rod extends, thus forcing the special nut against one of the outside plates of the specimen. This action puts tension in the threaded rod and applies a clamping force to the specimen, thereby simulating the effect of a pretensioned bolt. If the diameter of the centerhole ram is greater than 1 in., additional plate washers will be necessary at the ends of the ram. The clamping force system shall have a capability to apply a load of at least 49 kips and shall maintain this load during the test with an accuracy of 0.5 kips.

Commentary:
The slip coefficient can be easily determined using the hydraulic bolt test setup included in this Specification. The clamping force system simulates the clamping action of a pretensioned *high-strength bolt*. The centerhole ram applies a clamping force to the specimen, simulating that due to a pretensioned bolt.

A3.1.2. Compressive Load System: A compressive load shall be applied to the specimen until slip occurs. This compressive load shall be applied with a compression test machine or a reaction frame using a hydraulic loading device. The loading device and the necessary supporting elements shall be able to support a force of 120 kips. The compression loading system shall have a minimum accuracy of 1 percent of the slip load.

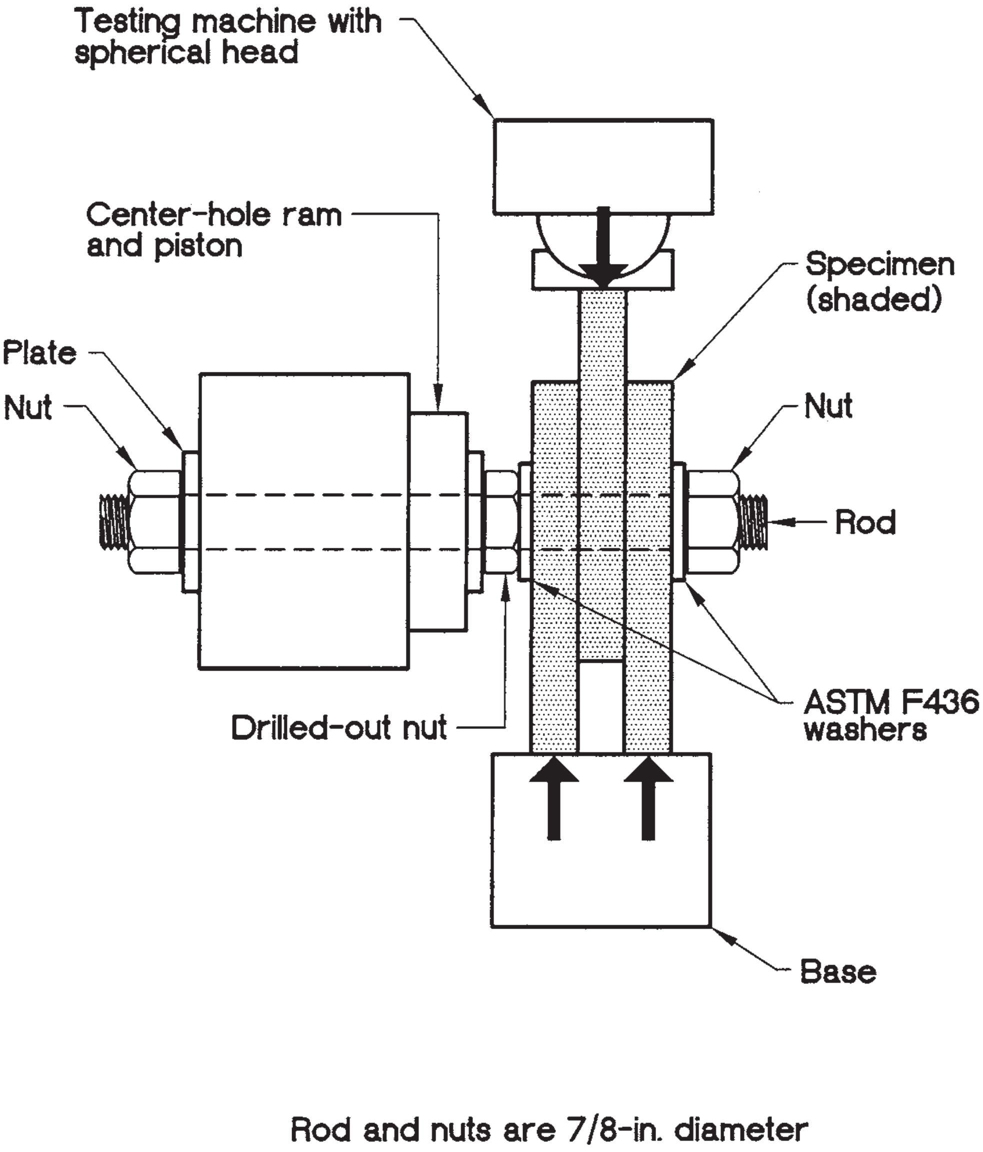


Figure A-3. Compression slip test setup.

A3.2. Instrumentation

A3.2.1. Clamping Force: The clamping force shall be measured within 0.5 kips. This is accomplished by measuring the pressure in the calibrated ram or placing a load cell in series with the ram.

A3.2.2. Compression Load: The compression load shall be measured during the test by direct reading from a compression testing machine, a load cell in series with the specimen and the compression loading device or pressure readings on a calibrated compression ram.

A3.2.3. Slip Deformation: The displacement of the center plate relative to the two outside plates shall be measured. This displacement, called “slip” for simplicity, shall be the average or that which occurs at the centerline of the specimen. This can be accomplished by using the average of two gages placed on the two exposed edges of the specimen or by monitoring the movement of the loading head relative to the base. If the latter method is used, due regard shall be taken for any slack that may be present in the loading system prior to application of the load. Deflections shall be measured by dial gages or any other calibrated device that has an accuracy of at least 0.001 in.

A3.3. Test Procedure

The specimen shall be installed in the test setup as shown in Figure A3. Before the hydraulic clamping force is applied, the individual plates shall be positioned so that they are in, or close to, full bearing contact with the ⅞ in. threaded rod in a direction that is opposite to the planned compressive loading to ensure obvious slip deformation. Care shall be taken in positioning the two outside plates so that the specimen is perpendicular to the base with both plates in contact with the base. After the plates are positioned, the centerhole ram shall be engaged to produce a clamping force of 49 kips. The applied clamping force shall be maintained within ±0.5 kips during the test until slip occurs.

The spherical head of the compression loading machine shall be brought into contact with the center plate of the specimen after the clamping force is applied. The spherical head or other appropriate device ensures concentric loading. When 1 kip or less of compressive load is applied, the slip gages shall be engaged or attached. The purpose of engaging the deflection gage(s), after a slight load is applied, is to eliminate initial specimen settling deformation from the slip reading.

When the slip gages are in place, the compression load shall be applied at a rate that does not exceed 25 kips per minute nor 0.003 in. of slip displacement per minute until the slip load is reached. The test should be terminated when a slip of 0.05 in. or greater is recorded. The load-slip relationship should preferably be monitored continuously on an X-Y plotter throughout the test, but in lieu of continuous data, sufficient load-slip data shall be recorded to evaluate the slip load defined below.

A3.4. Slip Load

Typical load-slip response is shown in Figure A4. Three types of curves are usually observed and the slip load associated with each type is defined as follows:

Curve (a)	Slip load is the maximum load, provided this maximum occurs before a slip of 0.02 in. is recorded.
Curve (b)	Slip load is the load at which the slip rate increases suddenly.
Curve (c)	Slip load is the load corresponding to a deformation of 0.02 in. This definition applies when the load vs. slip curves show a gradual change in response.

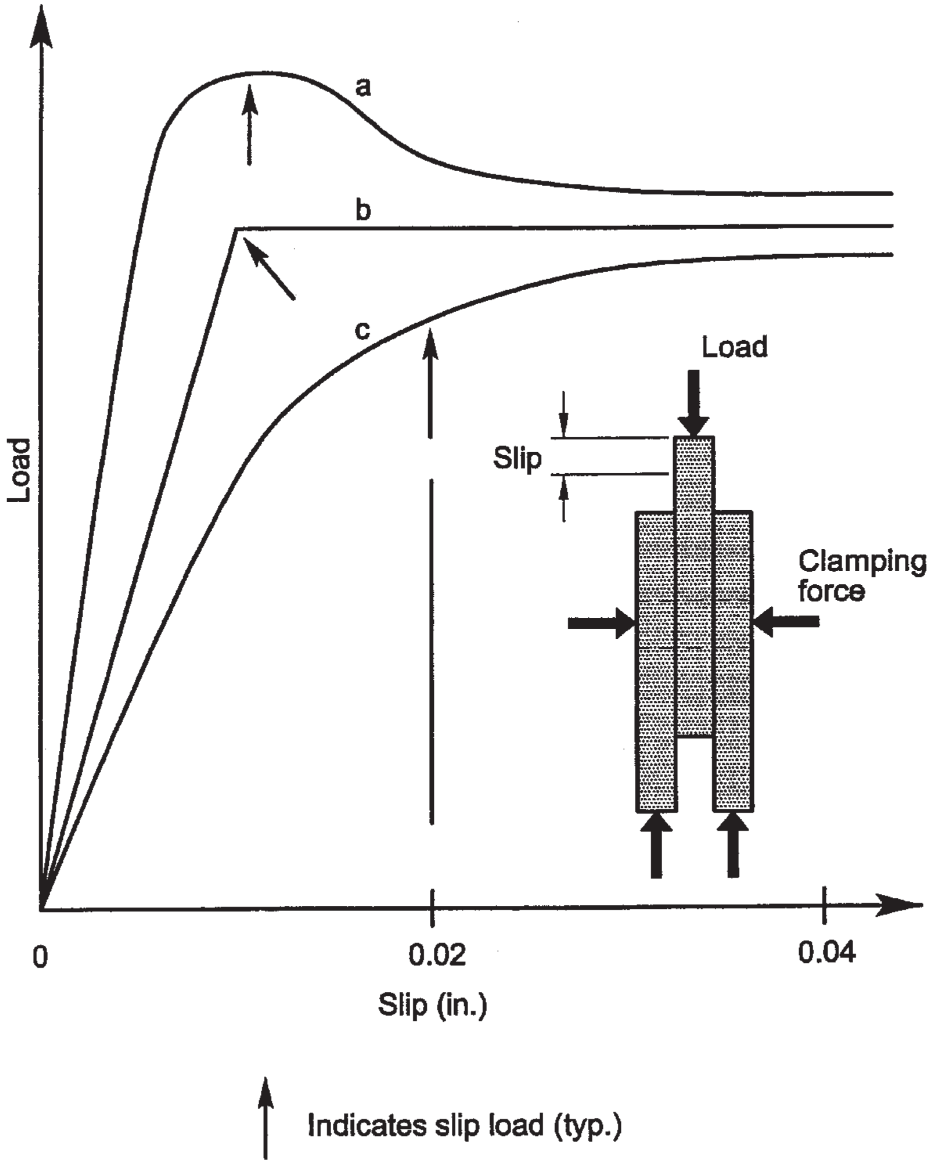


Figure A-4. Definition of slip load.

A3.5. Slip Coefficient

The slip coefficient for an individual specimen k_s shall be calculated as follows: The *mean slip coefficient* μ for one set of five specimens shall be reported.

$$k_s = \frac{slip\ load}{2 \times\ clamping\ force} \qquad \text{(Equation A3.1)}$$

A3.6. Alternative Test Methods

Alternative test methods to determine slip are permitted, provided the accuracy of load measurement and clamping satisfies the conditions presented in the previous sections. For example, the slip load may be determined from a tension-type test setup rather than the compression-type test setup as long as the contact surface area per bolt of the test specimen is the same as that shown in Figure A1. The clamping force of at least 49 kips may be applied by any means, provided the force can be established within ± 1 percent.

Commentary:

Alternative test procedures and specimens may be used as long as the accuracy of load measurement and specimen geometry are maintained as prescribed. For example, strain-gaged bolts can usually provide the desired accuracy. However, bolts that are pretensioned by the turn-of-nut, calibrated wrench, alternative-design fastener, or direct-tension-indicator pretensioning method usually show too much variation to meet the ± 1 percent requirement of the slip test.

SECTION A4. TENSION CREEP TEST

The test method outlined is intended to ensure that the coating will not undergo significant creep deformation under sustained service loading. The test also indicates the loss in clamping force in the bolt due to the compression or creep of the coating. Three replicate specimens are to be tested.

Commentary:
The creep deformation of the bolted *joint* under the applied shear loading is also an important characteristic and a function of the coating applied. Thicker coatings tend to creep more than thinner coatings. Rate of creep deformation increases as the applied load approaches the slip load. Extensive testing has shown that the rate of creep is not constant with time, rather it decreases with time. After about 1,000 hours of loading, the additional creep deformation is negligible.

A4.1. Test Setup

Tension-type specimens, as shown in Figure A2, are to be used. The replicate specimens are to be linked together in a single chain-like arrangement, using loose pin bolts, so the same load is applied to all specimens. The specimens shall be assembled so the specimen plates are bearing against the bolt in a direction opposite to the applied tension loading. Care shall be taken in the assembly of the specimens to ensure the centerline of the holes used to accept the pin bolts is in line with the bolts used to assemble the *joint*. The load level, specified in Section A4.2, shall be maintained constant within ±1 percent by springs, load maintainers, servo controllers, dead weight or other suitable equipment. The bolts used to clamp the specimens together shall be ⅞ in. diameter ASTM A490 bolts. All bolts shall come from the same *lot*.

The clamping force in the bolts shall be a minimum of 49 kips. The clamping force shall be determined by calibrating the bolt force with bolt elongation, if standard bolts are used. Alternatively, special *fastener assemblies* that control the clamping force by other means, such as calibrated bolt torque or strain gages, are permitted. A minimum of three bolt calibrations shall be performed using the technique selected for bolt force determination. The average of the three-bolt calibration shall be calculated and reported. The method of measuring bolt force shall ensure the clamping force is within ±2 kips of the average value.

The relative slip between the outside plates and the center plates shall be measured to an accuracy of 0.001 in. These slips are to be measured on both sides of each specimen.

A4.2. Test Procedure

The load to be placed on the creep specimens is the service load permitted for ⅞ in. diameter ASTM A490 bolts in *slip-critical joints* in Section 5 for the particular slip coefficient category under consideration. The load shall be placed on the specimen and held for 1,000 hours. The creep deformation of a specimen is calculated using the average reading of the two displacements on either side of the specimen. The difference between the average after 1,000 hours and the initial average reading taken within one-half hour after loading the specimens is defined as the creep deformation of the specimen. This value shall be reported for each specimen. If the creep deformation of any specimen exceeds 0.005 in., the coating has failed the test for the slip coefficient used. The coating may be retested using new specimens in accordance with this Section at a load corresponding to a lower value of slip coefficient.

If the value of creep deformation is less than 0.005 in. for all specimens, the specimens shall be loaded in tension to a load that is equal to the average clamping force times the design slip coefficient times 2, since there are two slip planes. The average slip deformation that occurs at this load shall be less than 0.015 in. for the three specimens. If the deformation is greater than this value, the coating is considered to have failed to meet the requirements for the particular *mean slip coefficient* used. The value of deformation for each specimen shall be reported.

Commentary:
See Commentary in Section A1.1

APPENDIX B. ALLOWABLE STRESS DESIGN (ASD) ALTERNATIVE

As an alternative to the load and resistance factor design provisions given in Sections 1 through 10, the following allowable stress design provisions are permitted. The provisions in Sections 1 through 10 in this Specification shall apply to ASD, except as follows:

B1.2 Loads, Load Factors and Load Combinations

The design and construction of the structure shall conform to an applicable allowable stress design specification for steel structures. When permitted in the applicable building code or specification, the allowable stresses in Section B5 are permitted to be increased to account for the effects of multiple transient loads in combination. When a load reduction factor is used to account for the effects of multiple transient loads in combination, the allowable stresses in Section B5 shall not be increased.

Commentary:

Although loads, load factors and load combinations are not explicitly specified in this Specification, the allowable stresses herein are based upon those specified in ASCE 7. When the design is governed by other load criteria, the allowable stresses specified herein shall be adjusted as appropriate.

SECTION B5. LIMIT STATES IN BOLTED JOINTS

The allowable shear strength and the allowable tensile strength of bolts shall be determined in accordance with Section B5.1. The interaction of combined shear and tension on bolts shall be limited in accordance with Section B5.2. The allowable bearing strength of the connected parts at bolt holes shall be determined in accordance with Section B5.3. Each of these allowable strengths shall be equal to or greater than the effect of the service loads. The axial load in bolts that are subject to tension or combined shear and tension shall be calculated with consideration of the externally applied tensile load and any additional tension resulting from *prying action* produced by deformation of the connected parts.

When slip resistance is required at the *faying surfaces* subject to shear or combined shear and tension, the slip resistance determined in accordance with Section B5.4 shall be equal to or greater than the effect of the service loads. In addition, the strength requirements in Sections B5.1, B5.2 and B5.3 shall also be met.

When bolts are subject to cyclic application of axial tension, the allowable stress determined in accordance with Section B5.5 shall be equal to or greater than the stress due to the effect of the service loads, including any additional tension resulting from *prying action* produced by deformation of the connected parts. In addition, the strength requirements in Sections B5.1, B5.2 and B5.3 shall also be met.

B5.1. Allowable Shear and Tensile Stresses

Shear and tensile strengths shall not be reduced by the installed bolt pretension. For *joints*, the allowable strength shall be based upon the allowable shear and tensile stresses of the individual bolts and shall be taken as the sum of the allowable strengths of the individual bolts.

The allowable shear strength or allowable tensile strength for an ASTM A325, A490 or F1852 bolt is R_a, where:

$$R_a = F_a A_b \quad \text{(Equation B5.1)}$$

where

R_a = allowable shear strength per shear plane or allowable tensile strength of a bolt, kips;

F_a = allowable stress from Table B5.1 for the appropriate applied load conditions, ksi, adjusted for the presence of fillers or shims as required below; and,

A_b = cross-sectional area based upon the nominal diameter of bolt, in.^{2}

When a bolt that carries load passes through fillers or shims in a shear plane that are equal to or less than ¼ in. thick, F_a from Table B5.1 shall be used without reduction. When a bolt that carries load passes through fillers or shims that are greater than ¼ in. thick, one of the following requirements shall apply:

(1) For fillers or shims that are equal to or less than ¾ in. thick, F_a from Table B5.1 shall be multiplied by the factor $[1 - 0.4(t' - 0.25)]$, where t' is the total thickness of fillers or shims, in., up to ¾ in.;

(2) The fillers or shims shall be extended beyond the *joint* and the filler extension shall be secured with enough bolts to uniformly distribute the total force in the connected element over the combined cross-section of the connected element and the fillers or shims;

(3) The size of the *joint* shall be increased to accommodate a number of bolts that is equivalent to the total number required in (2) above; or

(4) The *joint* shall be designed as a *slip-critical joint*. The slip resistance of the *joint* shall not be reduced for the presence of fillers or shims.

B5.2. Combined Shear and Tension Stress

When combined shear and tension loads are transmitted by an ASTM A325, A490 or F1852 bolt, the bolt shall be proportioned so that the tensile stress F_t, ksi, on the cross-sectional area based upon the nominal diameter of bolt A_b produced by forces applied to the connected parts, shall not exceed the values computed from the equations in Table B5.2, where f_v, the shear stress produced by the same forces, shall not exceed the value for shear determined in accordance with the requirements in Section B5.1.

Table B5.1. Allowable Stress in Bolts

Applied Load Condition		Allowable Stress F_a, ksi	
		ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Tension [a]	Static	44	54
	Fatigue	See Section B5.5	
Shear [a,b]	Threads included in shear plane	21	28
	Threads excluded from shear plane	30	40

[a] Except as required in Section B5.2.

[b] In shear *connections* that transmit axial force and have length between extreme bolts measured parallel to the line of force exceeds 50 in., tabulated values shall be reduced by 20 percent.

Table B5.2. Allowable Tensile Stress F_t for Bolts Subject to Combined Shear and Tension

Thread Condition	Allowable Tensile Stress F_t, ksi	
	ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Threads included in Shear plane	$\sqrt{(44)^2 - 4.39 f_v^2}$	$\sqrt{(54)^2 - 3.71 f_v^2}$
Threads excluded From shear plane	$\sqrt{(44)^2 - 2.15 f_v^2}$	$\sqrt{(54)^2 - 1.82 f_v^2}$

B5.3. Allowable Bearing at Bolt Holes

For *joints*, the allowable bearing strength shall be taken as the sum of the strengths of the connected material at the individual bolt holes.

The allowable bearing strength of the connected material at a standard bolt hole, oversized bolt hole, short-slotted bolt hole independent of the direction of loading or long-slotted bolt hole with the slot parallel to the direction of the bearing load is R_a, where:

(1) when deformation of the bolt hole at service load is a design consideration;

$$R_a = 0.6 L_c t F_u \le 1.2 d_b t F_u \qquad \text{(Equation B5.2)}$$

(2) when deformation of the bolt hole at service load is not a design consideration;

$$R_a = 0.75 L_c t F_u \le 1.5 d_b t F_u \qquad \text{(Equation B5.3)}$$

The allowable bearing strength of the connected material at a long-slotted bolt hole with the slot perpendicular to the direction of the bearing load is R_a, where:

$$R_a = 0.5 L_c t F_u \le d_b t F_u \qquad \text{(Equation B5.4)}$$

In Equations B5.2, B5.3 and B5.4,

R_a = allowable bearing strength of the connected material, kips;
F_u = specified minimum tensile strength (per unit area) of the connected material, ksi;
L_c = clear distance, in the direction of load, between the edge of the hole and the edge of the adjacent hole or the edge of the material, in.;
d_b = nominal bolt diameter, in.; and,
t = thickness of the connected material, in.

B5.4. Allowable Slip Resistance

The allowable slip resistance is R_a, where:

$$R_a = H \mu D T_m N_b \left(1 - \frac{T}{D T_m N_b}\right) \qquad \text{(Equation B5.5)}$$

where

H = 1.0 for standard holes
= 0.85 for oversized and short-slotted holes
= 0.70 for long-slotted holes perpendicular to the direction of load
= 0.60 for long-slotted holes parallel to the direction of load;
μ = *mean slip coefficient* for Class A, B or C faying surfaces, as applicable, or as established by testing in accordance with Appendix A (see Section 3.2.2(b))

Table B5.3. Allowable Stress for Fatigue Loading

Number of Cycles	Maximum Bolt Stress for Design at Service Loads[a], ksi	
	ASTM A325 or F1852 Bolt	ASTM A490 Bolt
Not more than 20,000	44	54
From 20,000 to 500,000	40	49
More than 500,000	31	38

[a] Including the effects of *prying action*, if any, but excluding the pretension.

= 0.33 for Class A *faying surfaces* (uncoated clean mill scale steel surfaces or surfaces with Class A coatings on blast cleaned steel)

= 0.50 for Class B surfaces (uncoated blast-cleaned steel surfaces or surfaces with Class B coatings on blast-cleaned steel)

= 0.35 for Class C surfaces (roughened hot-dip galvanized surfaces);

D = 0.80, a slip probability factor that reflects the distribution of actual slip coefficient values about the mean, the ratio of measured bolt tensile strength to the specified minimum values, and a slip probability level; the use of other values of *D* shall be approved by the *Engineer of Record*;

T_m = specified minimum bolt pretension (for pretensioned joints as specified in Table 8.1), kips;

N_b = number of bolts in the joint; and,

T = applied service load in tension (tensile component of applied service load for combined shear and tension loading), kips

= zero if the joint is subject to shear only

B5.5. Tensile Fatigue

The tensile stress in the bolt that results from the cyclic application of externally applied service loads and the prying force, if any, but not the pretension, shall not exceed the stress in Table B5.3. The nominal diameter of the bolt shall be used in calculating the bolt stress. The connected parts shall be proportioned so that the calculated prying force does not exceed 30 percent of the externally applied load. *Joints* that are subject to tensile fatigue loading shall be pretensioned in accordance with Section 4.2 or slip-critical in accordance with Section 4.

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Code of Standard Practice for Steel Buildings and Bridges

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Prepared by the American Institute of Steel Construction, Inc. under the direction of the AISC Committee on the Code of Standard Practice and issued by the AISC Board of Directors.

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PREFACE

As in any industry, trade practices have developed among those that are involved in the design, purchase, fabrication and erection of structural steel. This Code provides a useful framework for a common understanding of the acceptable standards when contracting for structural steel. As such, it is useful for owners, architects, engineers, general contractors, construction managers, fabricators, steel detailers, erectors and others that are associated with construction in structural steel. Unless specific provisions to the contrary are contained in the contract documents, the existing trade practices that are contained herein are considered to be the standard custom and usage of the industry and are thereby incorporated into the relationships between the parties to a contract.

The Symbols and Glossary are an integral part of this Code. In many sections of this Code, a non-mandatory Commentary has been prepared to provide background and further explanation for the corresponding Code provisions. The user is encouraged to consult it.

Since the first edition of this Code was published in 1924, AISC has continuously surveyed the structural steel design community and construction industry to determine standard trade practices. Since then, this Code has been periodically updated to reflect new and changing technology and industry practices.

The 2000 edition was the fifth complete revision of this Code since it was first published. The 2005 edition is not a complete revision but does add several important changes and updates. It is the result of the deliberations of a fair and balanced Committee, the membership of which included six structural engineers, two architects, one code official, one general contractor, eight fabricators, one steel detailer, three erectors, two inspectors, and one attorney. The following changes have been made in this revision:

- The intent of Section 1.1 has been clarified with additional Commentary.
- Section 1.5.2 has been modified to better address Owner-established performance criteria.
- The intent of the first sentence in Section 1.8.2 has been clarified.
- The order of paragraphs in Section 3.3 has been reversed to highlight that discovered discrepancies must be reported for resolution.
- The requirements in Section 3.4 for scale of design drawings have been modified.
- A requirement has been added in Section 4.2 for identification of Shop and Erection Drawings. Additionally, a paragraph has been added in the Commentary to this section addressing the use of independent detailing services, and the paragraph addressing the submittal schedule has been modified.
- A paragraph has been added to the Commentary in Section 4.4 addressing Shop and Erection Drawings that are approved subject to corrections noted, as well as Shop and Erection Drawings that are not approved.

- Coverage has been added of the RFI process in Section 4.6. Concurrently, explicit mention of RFIs has been added in Sections 3.5 and 4.4.2. Additionally, definitions have been added in the Glossary of the terms RFI, Clarification and Revision.
- The requirements for material identification have been modified in Section 6.1. Compatible modifications have also been made in Section 5.1.1.
- The requirements in Section 6.4.5 have been expanded to address fabricated trusses specified without camber. Compatible additions have been made in Sections 7.13.1.2(g) and (h).
- Section 7.4 has been modified to change “building lines” to “lines”.
- The Established Column Line definition in the Glossary has been changed, the definition of the term Column Line has been changed, and the usage of these terms in Section 7.5.1 has been changed for consistency with these definitions.
- Additional Commentary has been provided in Section 7.10.1 to illustrate the required description of the lateral load resisting system.
- Explicit mention of Erection Bracing Drawings has been added in the Commentary to Section 7.10.3.
- The intent of Section 8.5.5 has been clarified.
- Item 9.2.2(d) has been modified to change “detailed overall length” to “overall length”.
- Appendix A has been added to explicitly allow the user of this Code to choose to use electronic means for the exchange of project information.

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GLOSSARY

The following terms are used in this Code. Where used, they are capitalized to alert the user that the term is defined in this Glossary.

AASHTO. American Association of State Highway and Transportation Officials.

Adjustable Items. See Section 7.13.1.3.

AESS. See Architecturally Exposed Structural Steel.

AISC. American Institute of Steel Construction, Inc.

Anchor Bolt. See Anchor Rod.

Anchor Rod. A mechanical device that is either cast or drilled and chemically adhered, grouted or wedged into concrete and/or masonry for the purpose of the subsequent attachment of Structural Steel.

Anchor-Rod Group. A set of Anchor Rods that receives a single fabricated Structural Steel shipping piece.

ANSI. American National Standards Institute.

Architect. The entity that is professionally qualified and duly licensed to perform architectural services.

Architecturally Exposed Structural Steel. See Section 10.

AREMA. American Railway Engineering and Maintenance of Way Association.

ASME. American Society of Mechanical Engineers.

ASTM. American Society for Testing and Materials.

AWS. American Welding Society.

Bearing Devices. Shop-attached base and bearing plates, loose base and bearing plates and leveling devices, such as leveling plates, leveling nuts and washers and leveling screws.

CASE. Council of American Structural Engineers.

Clarification. An interpretation, of the Design Drawings or Specifications that have been Released for Construction, made in response to an RFI or a note on an approval drawing and providing an explanation that neither revises the information that has been Released for Construction nor alters the cost or schedule of performance of the work.

the Code, this Code. This document, the AISC *Code of Standard Practice for Steel Buildings and Bridges* as adopted by the American Institute of Steel Construction, Inc.

Column line. The grid line of column centers set in the field based on the dimensions shown on the structural design drawings and using the building layout provided by the Owners Designated Representative for Construction. Column offsets are taken from the column line. The column line may be straight or curved as shown in the structural design drawings.

Connection. An assembly of one or more joints that is used to transmit forces between two or more members and/or connection elements.

Contract Documents. The documents that define the responsibilities of the parties that are involved in bidding, fabricating and erecting Structural Steel. These documents normally include the Design Drawings, the Specifications and the contract.

Design Drawings. The graphic and pictorial portions of the Contract Documents showing the design, location and dimensions of the work. These documents generally include plans, elevations, sections, details, schedules, diagrams and notes.

Embedment Drawings. Drawings that show the location and placement of items that are installed to receive Structural Steel.

EOR. See Structural Engineer of Record.

Engineer. See Structural Engineer of Record.

Engineer of Record. See Structural Engineer of Record.

Erection Bracing Drawings. Drawings that are prepared by the Erector to illustrate the sequence of erection, any requirements for temporary supports and the requirements for raising, bolting and/or welding. These drawings are in addition to the Erection Drawings.

Erection Drawings. Field-installation or member-placement drawings that are prepared by the Fabricator to show the location and attachment of the individual shipping pieces.

Erector. The entity that is responsible for the erection of the Structural Steel.

Established Column Line. The actual field line that is most representative of the erected column centers along a line of columns placed using the dimensions shown in the structural Design Drawings and the lines and bench marks established by the Owner's Designated Representative for Construction, to be used in applying the erection tolerances given in this Code for column shipping pieces.

Fabricator. The entity that is responsible for fabricating the Structural Steel.

Hazardous Materials. Components, compounds or devices that are either encountered during the performance of the contract work or incorporated into it containing substances that, not withstanding the application of reasonable care, present a threat of harm to persons and/or the environment.

Inspector. The Owner's testing and inspection agency.

MBMA. Metal Building Manufacturers Association.

Mill Material. Steel mill products that are ordered expressly for the requirements of a specific project.

Owner. The entity that is identified as such in the Contract Documents.

Owner's Designated Representative for Construction. The Owner or the entity that is responsible to the Owner for the overall construction of the project, including its planning, quality and completion. This is usually the general contractor, the construction manager or similar authority at the job site.

Owner's Designated Representative for Design. The Owner or the entity that is responsible to the Owner for the overall structural design of the project, including the Structural Steel frame. This is usually the Structural Engineer of Record.

Plans. See Design Drawings.

RCSC. Research Council on Structural Connections.

Released for Construction. The term that describes the status of Contract Documents that are in such a condition that the Fabricator and the Erector can rely upon them for the performance of their work, including the ordering of material and the preparation of Shop and Erection Drawings.

Revision. An instruction or directive providing information that differs from information that has been Released for Construction. A Revision may, but does not always, impact the cost or schedule of performance of the work.

RFI. A written request for information or clarification generated during the construction phase of the project.

SER. See Structural Engineer of Record.

Shop Drawings. Drawings of the individual Structural Steel shipping pieces that are to be produced in the fabrication shop.

SJI. Steel Joist Institute.

Specifications. The portion of the Contract Documents that consists of the written requirements for materials, standards and workmanship.

SSPC. SSPC: The Society for Protective Coatings, which was formerly known as the Steel Structures Painting Council.

Standard Structural Shapes. Hot-rolled W-, S-, M- and HP-shapes, channels and angles listed in ASTM A6/A6M; structural tees split from the hot-rolled W-, S- and M-shapes listed in ASTM A6/A6M; hollow structural sections produced to ASTM A500, A501, A618 or A847; and, steel pipe produced to ASTM A53/A53M.

Steel Detailer. The entity that produces the Shop and Erection Drawings.

Structural Engineer of Record. The licensed professional who is responsible for sealing the Contract Documents, which indicates that he or she has performed or supervised the analysis, design and document preparation for the structure and has knowledge of the load-carrying structural system.

Structural Steel. The elements of the structural frame as given in Section 2.1.

Tier. The Structural Steel framing defined by a column shipping piece.

Weld Show-Through. In Architecturally Exposed Structural Steel, visual indication of the presence of a weld or welds on the side of the member opposite the weld.

CODE OF STANDARD PRACTICE FOR STEEL BUILDINGS AND BRIDGES

SECTION 1. GENERAL PROVISIONS

1.1. Scope

In the absence of specific instructions to the contrary in the Contract Documents, the trade practices that are defined in this Code shall govern the fabrication and erection of Structural Steel.

Commentary:
The practices defined in this Code are the commonly accepted standards of custom and usage for Structural Steel fabrication and erection, which generally represent the most efficient approach. This Code is not intended to define a professional standard of care for the Owners Designated Representative for Design, change the duties and responsibilities of the Owner, Contractor, Architect or Structural Engineer from those set forth in the Contract Documents, or assign to the Owner, Architect or Structural Engineer any duty or authority to undertake responsibility inconsistent with the provisions of the Contract Documents.

This Code is not applicable to steel joists or metal building systems, which are addressed by SJI and MBMA, respectively.

1.2. Referenced Specifications, Codes and Standards

The following documents are referenced in this Code:

AASHTO Specification—The 2004 AASHTO *LRFD Bridge Design Specifications*, 3rd Edition, with interims, or the 2002 AASHTO *Standard Specifications for Highway Bridges*, 17th Edition, with interims.

AISC Manual of Steel Construction—The AISC *Manual of Steel Construction, 13th Edition.*

AISC Seismic Provisions—The AISC *Seismic Provisions for Structural Steel Buildings*, March 9, 2005.

AISC Specification—The AISC *Specification for Structural Steel Buildings*, March 9, 2005.

ANSI/ASME B46.1—ANSI/ASME B46.1-95, Surface Texture (Surface Roughness, Waviness and Lay).

AREMA Specification—The 1999 AREMA *Manual for Railway Engineering, Volume II—Structures, Chapter 15*.

ASTM A6/A6M—04a, *Standard Specification for General Requirements for Rolled Structural Steel Bars, Plates, Shapes, and Sheet Piling.*

ASTM A53/A53M—02, *Standard Specification for Pipe, Steel, Black and Hot-Dipped, Zinc-Coated, Welded and Seamless*.

ASTM A325—04, *Standard Specification for Structural Bolts, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength*.

ASTM A325M—04, *Standard Specification for High-Strength Bolts for Structural Steel Joints (Metric)*.

ASTM A490—04, *Standard Specification for Heat-Treated Steel Structural Bolts, 150 ksi Minimum Tensile Strength*.

ASTMA490M—04, *Standard Specification for High-Strength Steel Bolts, Classes 10.9 and 10.9.3, for Structural Steel Joints (Metric)*.

ASTM A500—03a, *Standard Specification for Cold-Formed Welded and Seamless Carbon Steel Structural Tubing in Rounds and Shapes*. No metric equivalent exists.

ASTM A501—01, *Standard Specification for Hot-Formed Welded and Seamless Carbon Steel Structural Tubing*. No metric equivalent exists.

ASTM A618—04, *Standard Specification for Hot-Formed Welded and Seamless High-Strength Low-Alloy Structural Tubing*. No metric equivalent exists.

ASTM A847—99a(2003), *Standard Specification for Cold-Formed Welded and Seamless High-Strength, Low-Alloy Structural Tubing with Improved Atmospheric Corrosion Resistance*. No metric equivalent exists.

ASTM F1852/F1852M—04, *Standard Specification for "Twist-Off" Type Tension Control Structural Bolt/Nut/Washer Assemblies, Steel, Heat Treated, 120/105 ksi Minimum Tensile Strength.*

AWS D1.1—The AWS D1.1 *Structural Welding Code—Steel*, 2004.

CASE Document 11—*An Agreement Between Structural Engineer of Record and Contractor for Transfer of Computer Aided Drafting (CAD) files on Electronic Media*, 2000

CASE Document 962—The *National Practice Guidelines for the Structural Engineer of Record*, Fourth Edition, 2000.

RCSC Specification—The *Specification for Structural Joints Using ASTM A325 or A490 Bolts*, 2004.

SSPC SP2—SSPC *Surface Preparation Specification No. 2, Hand Tool Cleaning*, 2004.

SSPC SP6—SSPC *Surface Preparation Specification No. 6, Commercial Blast Cleaning*, 2004.

1.3. Units

In this Code, the values stated in either U.S. customary units or metric units shall be used. Each system shall be used independently of the other.

Commentary:

In this Code, dimensions, weights and other measures are given in U.S. customary units with rounded or rationalized metric-unit equivalents in brackets. Because the values stated in each system are not exact equivalents, the selective combination of values from each of the two systems is not permitted.

1.4. Design Criteria

For buildings, in the absence of other design criteria, the provisions in the AISC Specification shall govern the design of the Structural Steel. For bridges, in the absence of other design criteria, the provisions in the AASHTO Specification and AREMA Specification shall govern the design of the Structural Steel, as applicable.

1.5. Responsibility for Design

1.5.1. When the Owner's Designated Representative for Design provides the design, Design Drawings and Specifications, the Fabricator and the Erector are not responsible for the suitability, adequacy or building-code conformance of the design.

1.5.2. When the Owner enters into a direct contract with the Fabricator to both design and fabricate an entire, completed steel structure, the Fabricator shall be responsible for the suitability, adequacy, conformance with Owner-established performance criteria, and building-code conformance of the Structural Steel design. The Owner shall be responsible for the suitability, adequacy and building-code conformance of the non-Structural Steel elements and shall establish the performance criteria for the Structural Steel frame.

1.6. Patents and Copyrights

The entity or entities that are responsible for the specification and/or selection of proprietary structural designs shall secure all intellectual property rights necessary for the use of those designs.

1.7. Existing Structures

1.7.1. Demolition and shoring of any part of an existing structure are not within the scope of work that is provided by either the Fabricator or the Erector. Such demolition and shoring shall be performed in a timely manner so as not to interfere with or delay the work of the Fabricator and the Erector.

1.7.2. Protection of an existing structure and its contents and equipment, so as to prevent damage from normal erection processes, is not within the scope of work that is provided by either the Fabricator or the Erector. Such protection shall be performed in a timely manner so as not to interfere with or delay the work of the Fabricator or the Erector.

1.7.3. Surveying or field dimensioning of an existing structure is not within the scope of work that is provided by either the Fabricator or the Erector. Such surveying or field dimensioning, which is necessary for the completion of Shop and Erection Drawings and fabrication, shall be performed and furnished to the Fabricator in a timely manner so as not to interfere with or delay the work of the Fabricator or the Erector.

1.7.4. Abatement or removal of Hazardous Materials is not within the scope of work that is provided by either the Fabricator or the Erector. Such abatement or removal shall be performed in a timely manner so as not to interfere with or delay the work of the Fabricator and the Erector.

1.8. Means, Methods and Safety of Erection

1.8.1. The Erector shall be responsible for the means, methods and safety of erection of the Structural Steel frame.

1.8.2. The Structural Engineer of Record shall be responsible for the structural adequacy of the design of the structure in the completed project. The Structural Engineer of Record shall not be responsible for the means, methods and safety of erection of the Structural Steel frame. See also Sections 3.1.4 and 7.10.

SECTION 2. CLASSIFICATION OF MATERIALS

2.1. Definition of Structural Steel

Structural Steel shall consist of the elements of the structural frame that are shown and sized in the structural Design Drawings, essential to support the design loads and described as:

Anchor Rods that will receive Structural Steel.
Base plates.
Beams, including built-up beams, if made from Standard Structural Shapes and/or plates.
Bearing plates.
Bearings of steel for girders, trusses or bridges.
Bracing, if permanent.
Canopy framing, if made from Standard Structural Shapes and/or plates.
Columns, including built-up columns, if made from Standard Structural Shapes and/or plates.
Connection materials for framing Structural Steel to Structural Steel.
Crane stops, if made from Standard Structural Shapes and/or plates.
Door frames, if made from Standard Structural Shapes and/or plates and if part of the Structural Steel frame.
Edge angles and plates, if attached to the Structural Steel frame or steel (open-web) joists.
Embedded Structural Steel parts, other than bearing plates, that will receive Structural Steel.
Expansion joints, if attached to the Structural Steel frame.
Fasteners for connecting Structural Steel items: permanent shop bolts, nuts and washers; shop bolts, nuts and washers for shipment; field bolts, nuts and washers for permanent Connections; and, permanent pins.
Floor-opening frames, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame or steel (open-web) joists.
Floor plates (checkered or plain), if attached to the Structural Steel frame.
Girders, including built-up girders, if made from Standard Structural Shapes and/or plates.
Girts, if made from Standard Structural Shapes.
Grillage beams and girders.
Hangers, if made from Standard Structural Shapes, plates and/or rods and framing Structural Steel to Structural Steel.
Leveling nuts and washers.
Leveling plates.
Leveling screws.
Lintels, if attached to the Structural Steel frame.
Marquee framing, if made from Standard Structural Shapes and/or plates.

Machinery supports, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame.
Monorail elements, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame.
Posts, if part of the Structural Steel frame.
Purlins, if made from Standard Structural Shapes.
Relieving angles, if attached to the Structural Steel frame.
Roof-opening frames, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame or steel (open-web) joists.
Roof-screen support frames, if made from Standard Structural Shapes.
Sag rods, if part of the Structural Steel frame and connecting Structural Steel to Structural Steel.
Shear stud connectors, if specified to be shop attached.
Shims, if permanent.
Struts, if permanent and part of the Structural Steel frame.
Tie rods, if part of the Structural Steel frame.
Trusses, if made from Standard Structural Shapes and/or built-up members.
Wall-opening frames, if made from Standard Structural Shapes and/or plates and attached to the Structural Steel frame.
Wedges, if permanent.

Commentary:
The Fabricator normally fabricates the items listed in Section 2.1. Such items must be shown, sized and described in the structural Design Drawings. Bracing includes vertical bracing for resistance to wind and seismic load and structural stability, horizontal bracing for floor and roof systems and permanent stability bracing for components of the Structural Steel frame.

2.2. Other Steel, Iron or Metal Items

Structural Steel shall not include other steel, iron or metal items that are not generally described in Section 2.1, even where such items are shown in the structural Design Drawings or are attached to the Structural Steel frame. Other steel, iron or metal items include but are not limited to:

Bearings, if non-steel.
Cables for permanent bracing or suspension systems.
Castings.
Catwalks.
Chutes.
Cold-formed steel products.
Cold-rolled steel products, except those that are specifically covered in the AISC Specification.
Corner guards.

Crane rails, splices, bolts and clamps.
Crane stops, if not made from Standard Structural Shapes or plates.
Door guards.
Embedded steel parts, other than bearing plates, that do not receive Structural Steel or that are embedded in precast concrete.
Expansion joints, if not attached to the Structural Steel frame.
Flagpole support steel.
Floor plates (checkered or plain), if not attached to the Structural Steel frame.
Forgings.
Gage-metal products.
Grating.
Handrail.
Hangers, if not made from Standard Structural Shapes, plates and/or rods or not framing Structural Steel to Structural Steel.
Hoppers.
Items that are required for the assembly or erection of materials that are furnished by trades other than the Fabricator or Erector.
Ladders.
Lintels, if not attached to the Structural Steel frame.
Masonry anchors.
Miscellaneous metal.
Ornamental metal framing.
Pressure vessels.
Reinforcing steel for concrete or masonry.
Relieving angles, if not attached to the Structural Steel frame.
Roof screen support frames, if not made from Standard Structural Shapes.
Safety cages.
Shear stud connectors, if specified to be field installed.
Stacks.
Stairs.
Steel deck.
Steel (open-web) joists.
Steel joist girders.
Tanks.
Toe plates.
Trench or pit covers.

Commentary:

Section 2.2 includes many items that may be furnished by the Fabricator if contracted to do so by specific notation and detail in the Contract Documents. When such items are contracted to be provided by the Fabricator, coordination will normally be required between the Fabricator and other material suppliers and trades. The provisions in this Code are not intended to apply to items in Section 2.2.

In previous editions of this Code, provisions regarding who should normally furnish field-installed shear stud connectors and cold-formed steel deck support angles were included in Section 7.8. These provisions have been eliminated since field-installed shear stud connectors and steel deck support angles are not defined as Structural Steel in this Code.

SECTION 3. DESIGN DRAWINGS AND SPECIFICATIONS

3.1. Structural Design Drawings and Specifications

Unless otherwise indicated in the Contract Documents, the structural Design Drawings shall be based upon consideration of the design loads and forces to be resisted by the Structural Steel frame in the completed project.

The structural Design Drawings shall clearly show the work that is to be performed and shall give the following information with sufficient dimensions to accurately convey the quantity and nature of the Structural Steel to be fabricated:

(a) The size, section, material grade and location of all members;
(b) All geometry and working points necessary for layout;
(c) Floor elevations;
(d) Column centers and offsets;
(e) The camber requirements for members; and,
(f) The information that is required in Sections 3.1.1 through 3.1.6.

The Structural Steel Specification shall include any special requirements for the fabrication and erection of the Structural Steel.

The structural Design Drawings, Specifications and addenda shall be numbered and dated for the purposes of identification.

Commentary:

Contract Documents vary greatly in complexity and completeness. Nonetheless, the Fabricator and the Erector must be able to rely upon the accuracy and completeness of the Contract Documents. This allows the Fabricator and the Erector to provide the Owner with bids that are adequate and complete. It also enables the preparation of the Shop and Erection Drawings, the ordering of materials and the timely fabrication and erection of shipping pieces.

In some cases, the Owner can benefit when reasonable latitude is allowed in the Contract Documents for alternatives that can reduce cost without compromising quality. However, critical requirements that are necessary to protect the Owner's interest, that affect the integrity of the structure or that are necessary for the Fabricator and the Erector to proceed with their work must be included in the Contract Documents. Some examples of critical information include:

Standard specifications and codes that govern Structural Steel design and construction, including bolting and welding.
Material specifications.
Special material requirements to be reported on the certified mill test reports.
Welded-joint configuration.

Weld-procedure qualification.
Special requirements for work of other trades.
Final disposition of backing bars and runoff tabs.
Lateral bracing.
Stability bracing.
Connections or data for Connection selection and/or completion.
Restrictions on Connection types.
Column stiffeners (also known as continuity plates).
Column web doubler plates.
Bearing stiffeners on beams and girders.
Web reinforcement.
Openings for other trades.
Surface preparation and shop painting requirements.
Shop and field inspection requirements.
Non-destructive testing requirements, including acceptance criteria.
Special requirements on delivery.
Special erection limitations.
Identification of non-Structural Steel elements that interact with the Structural Steel frame to provide for the lateral stability of the Structural Steel frame (see Section 3.1.4).
Column differential shortening information.
Special fabrication and erection tolerances for AESS.
Special pay-weight provisions.

3.1.1. Permanent bracing, column stiffeners, column web doubler plates, bearing stiffeners in beams and girders, web reinforcement, openings for other trades and other special details, where required, shall be shown in sufficient detail in the structural Design Drawings so that the quantity, detailing and fabrication requirements for these items can be readily understood.

3.1.2. The Owner's Designated Representative for Design shall either show the complete design of the Connections in the structural Design Drawings or allow the Fabricator to select or complete the Connection details while preparing the Shop and Erection Drawings. When the Fabricator is allowed to select or complete the Connection details, the following information shall be provided in the structural Design Drawings:

(a) Any restrictions on the types of Connections that are permitted;
(b) Data concerning the loads, including shears, moments, axial forces and transfer forces, that are to be resisted by the individual members and their Connections, sufficient to allow the Fabricator to select or complete the Connection details while preparing the Shop and Erection Drawings;
(c) Whether the data required in (b) is given at the service-load level or the factored-load level; and,

(d) Whether LRFD or ASD is to be used in the selection or completion of Connection details.

When the Fabricator selects or completes the Connection details, the Fabricator shall utilize the requirements in the AISC Specification and the Contract Documents and submit the Connection details to the Owner's Designated Representative for Design for approval.

Commentary:
When the Owner's Designated Representative for Design shows the complete design of the Connections in the structural Design Drawings, the following information is included:

(a) All weld sizes and lengths;
(b) All bolt sizes, locations, quantities and grades;
(c) All plate and angle sizes, thicknesses and dimensions; and,
(d) All work point locations and related information.

The intent of this approach is that complete information necessary for Connection detailing, fabrication and erection is shown in the structural Design Drawings. The Steel Detailer will then be able to transfer this information to the Shop and Erection Drawings, applying it to the individual pieces being detailed.

When the Owner's Designated Representative for Design allows the Fabricator to select or complete the Connections, this is commonly done by referring to tables in the Contract Documents or in the AISC Manual of Steel Construction, or by schematically showing the types of Connections required in the structural Design Drawings. The Steel Detailer will then configure the Connections based upon the design loads and other information given in the structural Design Drawings. If the desired Connection is not covered in those tables, a detail of the "special" Connection should be contained in the structural Design Drawings. This detail should provide such information as weld sizes, plate thicknesses and quantities of bolts. However, there may be some geometry and dimensional information that the Steel Detailer must develop. The intent of this method is that the Steel Detailer will select the Connection materials and configuration from the referenced tables or complete the specific Connection configuration (i.e. dimensions, edge distances and bolt spacing) based upon the Connection details that are shown in the structural Design Drawings.

This method will require the skill of an experienced Steel Detailer, who is familiar with the AISC requirements for Connection configurations, capable and experienced in the use of the Connection tables in the AISC Manual of Steel Construction and capable of calculating dimensions and adapting a typical Connection detail to similar situations. Notations of loadings in the structural Design Drawings are only to facilitate selection of the Connections from the referenced tables. It is not the intent of this method that the Steel Detailer practice engineering.

If there are any restrictions as to the types of Connections to be used, particularly as it relates to simple shear Connections, it is required that these limitations be set forth in the structural Design Drawings and Specifications. There are a variety of Connections available in the AISC Manual of Steel Construction for a given situation. Preference for a particular type will vary between Fabricators and Erectors. Stating these limitations, if any, in the structural Design Drawings and Specifications will help to avoid repeated changes to the Shop and Erection Drawings due to the selection of a Connection that is not acceptable to the Owner's Designated Representative for Design, thereby avoiding additional cost and/or delay for the redrawing of the Shop and Erection Drawings.

The structural Design Drawings must indicate the method of design used as LRFD or ASD. In order to conform to the spirit of the AISC Specification, the Connections must be selected using the same method and the corresponding references.

3.1.3. When leveling plates are to be furnished as part of the contract requirements, their locations and required thickness and sizes shall be specified in the Contract Documents.

3.1.4. When the Structural Steel frame, in the completely erected and fully connected state, requires interaction with non-Structural Steel elements (see Section 2) for strength and/or stability, those non-Structural Steel elements shall be identified in the Contract Documents as required in Section 7.10.

Commentary:
Examples of non-Structural Steel elements include diaphragms made of steel deck, diaphragms made of concrete on steel deck and masonry and/or concrete shear walls.

3.1.5. When camber is required, the magnitude, direction and location of camber shall be specified in the structural Design Drawings.

Commentary:
For cantilevers, the specified camber may be up or down, depending upon the framing and loading.

3.1.6. Specific members or portions thereof that are to be left unpainted shall be identified in the Contract Documents. When shop painting is required, the painting requirements shall be specified in the Contract Documents, including the following information:

(a) The identification of specific members or portions thereof to be painted;
(b) The surface preparation that is required for these members;
(c) The paint specifications and manufacturer's product identification that are required for these members; and,
(d) The minimum dry-film shop-coat thickness that is required for these members.

Commentary:
Some members or portions thereof may be required to be left unpainted, such as those that will be in contact and acting compositely with concrete, or those that will receive spray-applied fire protection materials.

3.2. Architectural, Electrical and Mechanical Design Drawings and Specifications

All requirements for the quantities, sizes and locations of Structural Steel shall be shown or noted in the structural Design Drawings. The use of architectural, electrical and/or mechanical Design Drawings as a supplement to the structural Design Drawings is permitted for the purposes of defining detail configurations and construction information.

3.3. Discrepancies

When discrepancies exist between the Design Drawings and Specifications, the Design Drawings shall govern. When discrepancies exist between scale dimensions in the Design Drawings and the figures written in them, the figures shall govern. When discrepancies exist between the structural Design Drawings and the architectural, electrical or mechanical Design Drawings or Design Drawings for other trades, the structural Design Drawings shall govern.

When a discrepancy is discovered in the Contract Documents in the course of the Fabricator's work, the Fabricator shall promptly notify the Owner's Designated Representative for Construction so that the discrepancy can be resolved by the Owner's Designated Representative for Design. Such resolution shall be timely so as not to delay the Fabricator's work. See Sections 3.5 and 9.3.

Commentary:
While it is the Fabricator's responsibility to report any discrepancies that are discovered in the Contract Documents, it is not the Fabricator's responsibility to discover discrepancies, including those that are associated with the coordination of the various design disciplines. The quality of the Contract Documents is the responsibility of the entities that produce those documents.

3.4. Legibility of Design Drawings

Design Drawings shall be clearly legible and drawn to an identified scale that is appropriate to clearly convey the information.

Commentary:
Historically, the most commonly accepted scale for structural steel plans has been 1/8 in. per ft [10 mm per 1 000 mm]. There are, however, situations where a smaller or larger scale is appropriate. Ultimately, consideration must be given to the clarity of the drawing.

The scaling of the Design Drawings to determine dimensions is not an accepted practice for detailing the Shop and Erection Drawings. However, it should be remembered when preparing Design Drawings that scaling may be the only method available when early-submission drawings are used to determine dimensions for estimating and bidding purposes.

3.5. Revisions to the Design Drawings and Specifications

Revisions to the Design Drawings and Specifications shall be made either by issuing new Design Drawings and Specifications or by reissuing the existing Design Drawings and Specifications. In either case, all Revisions, including Revisions that are communicated through responses to RFIs or the annotation of Shop and/or Erection Drawings (see Section 4.4.2), shall be clearly and individually indicated in the Contract Documents. The Contract Documents shall be dated and identified by Revision number. Each Design Drawing shall be identified by the same drawing number throughout the duration of the project, regardless of the Revision. See also Section 9.3.

Commentary:
Revisions to the Design Drawings and Specifications can be made by issuing sketches and supplemental information separate from the Design Drawings and Specifications. These sketches and supplemental information become amendments to the Design Drawings and Specifications and are considered new Contract Documents. All sketches and supplemental information must be uniquely identified with a number and date as the latest instructions until such time as they may be superseded by new information.

When revisions are made by revising and re-issuing the existing structural Design Drawings and/or Specifications, a unique revision number and date must be added to those documents to identify that information as the latest instructions until such time as they may be superseded by new information. The same unique drawing number must identify each Design Drawing throughout the duration of the project so that revisions can be properly tracked, thus avoiding confusion and miscommunication among the various entities involved in the project.

When revisions are communicated through the annotation of Shop or Erection Drawings or contractor submissions, such changes must be confirmed in writing by one of the aforementioned methods. This written confirmation is imperative to maintain control of the cost and schedule of a project and to avoid potential errors in fabrication.

3.6. Fast-Track Project Delivery

When the fast-track project delivery system is selected, release of the structural Design Drawings and Specifications shall constitute a Release for Construction, regardless of the status of the architectural, electrical, mechanical and other interfacing designs and Contract Documents. Subsequent revisions, if any, shall be the responsibility of the Owner and shall be made in accordance with Sections 3.5 and 9.3.

Commentary:

The fast-track project delivery system generally provides for a condensed schedule for the design and construction of a project. Under this delivery system, the Owner elects to Release for Construction the structural Design Drawings and Specifications, which may be partially complete, at a time that may precede the completion of and coordination with architectural, mechanical, electrical and other design work and Contract Documents. The release of these structural Design Drawings and Specifications may also precede the release of the General Conditions and Division 1 Specifications.

Release of the structural Design Drawings and Specifications to the Fabricator for ordering of material constitutes a Release for Construction. Accordingly, the Fabricator and the Erector may begin their work based upon those partially complete documents. As the architectural, mechanical, electrical and other design elements of the project are completed, revisions may be required in design and/or construction. Thus, when considering the fast-track project delivery system, the Owner should balance the potential benefits to the project schedule with the project cost contingency that may be required to allow for these subsequent revisions.

SECTION 4. SHOP AND ERECTION DRAWINGS

4.1. Owner Responsibility

The Owner shall furnish, in a timely manner and in accordance with the Contract Documents, complete structural Design Drawings and Specifications that have been Released for Construction. Unless otherwise noted, Design Drawings that are provided as part of a contract bid package shall constitute authorization by the Owner that the Design Drawings are Released for Construction

Commentary:
When the Owner issues Released-for-Construction Design Drawings and Specifications, the Fabricator and the Erector rely on the fact that these are the Owner's requirements for the project. This release is required by the Fabricator prior to the ordering of material and the preparation and completion of Shop and Erection Drawings.

To ensure the orderly flow of material procurement, detailing, fabrication and erection activities, on phased construction projects, it is essential that designs are not continuously revised after they have been Released for Construction. In essence, once a portion of a design is Released for Construction, the essential elements of that design should be "frozen" to ensure adherence to the contract price and construction schedule. Alternatively, all parties should reach a common understanding of the effects of future changes, if any, as they affect scheduled deliveries and added costs.

4.2. Fabricator Responsibility

Except as provided in Section 4.5, the Fabricator shall produce Shop and Erection Drawings for the fabrication and erection of the Structural Steel and is responsible for the following:

(a) The transfer of information from the Contract Documents into accurate and complete Shop and Erection Drawings; and,
(b) The development of accurate, detailed dimensional information to provide for the fit-up of parts in the field.

Each Shop and Erection Drawing shall be identified by the same drawing number throughout the duration of the project and shall be identified by revision number and date, with each specific revision clearly identified.

When the Fabricator submits a request to change Connection details that are described in the Contract Documents, the Fabricator shall notify the Owner's Designated Representatives for Design and Construction in writing in advance of the submission of the Shop and Erection Drawings. The Owner's Designated Representative for Design shall review and approve or reject the request in a timely manner.

When requested to do so by the Owner’s Designated Representative for Design, the Fabricator shall provide to the Owner’s Designated Representatives for Design and Construction its schedule for the submittal of Shop and Erection Drawings so as to facilitate the timely flow of information between all parties.

Commentary:

The fabricator is permitted to use the services of independent detailers to produce shop and erection drawings and to perform other support services such as producing advanced bills of material and bolt summaries.

As the Fabricator develops the detailed dimensional information for production of the Shop and Erection Drawings, there may be discrepancies, missing information or conflicts discovered in the Contract Documents. See Section 3.3.

When the Fabricator intends to make a submission of alternative Connection details to those shown in the Contract Documents, the Fabricator must notify the Owner’s Designated Representatives for Design and Construction in advance. This will allow the parties involved to plan for the increased effort that may be required to review the alternative Connection details. In addition, the Owner will be able to evaluate the potential for cost savings and/or schedule improvements against the additional design cost for review of the alternative Connection details by the Owner’s Designated Representative for Design. This evaluation by the Owner may result in the rejection of the alternative Connection details or acceptance of the submission for review based upon cost savings, schedule improvements and/or job efficiencies.

The Owner’s Designated Representative for Design may request the Fabricator’s schedule for the submittal of shop and erection drawings. This process is intended to allow the parties to plan for the staffing demands of the submission schedule. The Contract Documents may address this issue in more detail. In the absence of the requirement to provide this schedule, none need be provided.

When the Fabricator provides a schedule for the submission of the Shop and Erection Drawings, it must be recognized that this schedule may be affected by revisions and the response time to requests for missing information or the resolution of discrepancies.

4.3. Use of CAD Files and/or Copies of Design Drawings

The Fabricator shall neither use nor reproduce any part of the Design Drawings as part of the Shop or Erection Drawings without the written permission of the Owner's Designated Representative for Design. When CAD files or copies of the Design Drawings are made available for the Fabricator’s use, the Fabricator shall accept this information under the following conditions:

(a) All information contained in the CAD files or copies of the Design Drawings shall be considered instruments of service of the Owner's Designated Representative for Design and shall not be used for other projects, additions to the project or the completion of the project by others. CAD files and copies of the Design Drawings shall remain the property of the Owner's Designated Representative for Design and in no case shall the transfer of these CAD files or copies of the Design Drawings be considered a sale.

(b) The CAD files or copies of the Design Drawings shall not be considered to be Contract Documents. In the event of a conflict between the Design Drawings and the CAD files or copies thereof, the Design Drawings shall govern;

(c) The use of CAD files or copies of the Design Drawings shall not in any way obviate the Fabricator's responsibility for proper checking and coordination of dimensions, details, member sizes and fit-up and quantities of materials as required to facilitate the preparation of Shop and Erection Drawings that are complete and accurate as required in Section 4.2; and,

(d) The Fabricator shall remove information that is not required for the fabrication or erection of the Structural Steel from the CAD files or copies of the Design Drawings.

Commentary:

With the advent of electronic media and the internet, electronic copies of Design Drawings are becoming readily available to the Fabricator. As a result, the Owner's Designated Representative for Design may have reduced control over the unauthorized use of the Design Drawings. There are many copyright and other legal issues to be considered.

The Owner's Designated Representative for Design may choose to make CAD files or copies of the Design Drawings available to the Fabricator, and may charge a service or licensing fee for this convenience. In doing so, a carefully negotiated agreement should be established to set out the specific responsibilities of both parties in view of the liabilities involved for both parties. For a sample contract, see CASE Document 11.

The CAD files and/or copies of the Design Drawings are provided to the Fabricator for convenience only. The information therein should be adapted for use only in reference to the placement of Structural Steel members during erection. The Fabricator should treat this information as if it were fully produced by the Fabricator and undertake the same level of checking and quality assurance. When amendments or revisions are made to the Contract Documents, the Fabricator must update this reference material.

When CAD files or copies of the Design Drawings are provided to the Fabricator, they often contain other information, such as architectural backgrounds or references to other Contract Documents. This additional material should be removed when producing Shop and Erection Drawings to avoid the potential for confusion.

4.4. Approval

Except as provided in Section 4.5, the Shop and Erection Drawings shall be submitted to the Owner's Designated Representatives for Design and Construction for review and approval. These drawings shall be returned to the Fabricator within 14 calendar days. Approved Shop and Erection Drawings shall be individually annotated by the Owner's Designated Representatives for Design and Construction as either approved or approved subject to corrections noted. When so required, the Fabricator shall subsequently make the corrections noted and furnish corrected Shop and Erection Drawings to the Owner's Designated Representatives for Design and Construction.

Commentary:

As used in this Code, the 14-day allotment for the return of Shop and Erection Drawings is intended to represent the Fabricator's portal-to-portal time. The intent in this Code is that, in the absence of information to the contrary in the Contract Documents, 14 days may be assumed for the purposes of bidding, contracting and scheduling. A submittal schedule is commonly used to facilitate the approval process.

If a Shop or Erection Drawing is approved subject to corrections noted, the Owner's Designated Representative for Design may or may not require that it be re-submitted for record purposes following correction. If a Shop or Erection Drawing is not approved, revisions must be made and the drawing re-submitted until approval is achieved.

4.4.1. Approval of the Shop and Erection Drawings, approval subject to corrections noted and similar approvals shall constitute the following:

(a) Confirmation that the Fabricator has correctly interpreted the Contract Documents in the preparation of those submittals;
(b) Confirmation that the Owner's Designated Representative for Design has reviewed and approved the Connection details shown on the Shop and Erection Drawings and submitted in accordance with Section 3.1.2, if applicable; and,
(c) Release by the Owner's Designated Representatives for Design and Construction for the Fabricator to begin fabrication using the approved submittals.

Such approval shall not relieve the Fabricator of the responsibility for either the accuracy of the detailed dimensions in the Shop and Erection Drawings or the general fit-up of parts that are to be assembled in the field.

The Fabricator shall determine the fabrication schedule that is necessary to meet the requirements of the contract.

Commentary:
When considering the current language in this Section, the Committee sought language that would parallel the practices of CASE. In CASE Document 962, CASE indicates that when the design of some element of the primary structural system is left to someone other than the Structural Engineer of Record, "…such elements, including connections designed by others, should be reviewed by the Structural Engineer of Record. He [or she] should review such designs and details, accept or reject them and be responsible for their effects on the primary structural system." Historically, this Code has embraced this same concept.

From the inception of this Code, AISC and the industry in general have recognized that only the Owner's Designated Representative for Design has all the information necessary to evaluate the total impact of Connection details on the overall structural design of the project. This authority has traditionally been exercised during the approval process for Shop and Erection Drawings. The Owner's Designated Representative for Design has thus retained responsibility for the adequacy and safety of the entire structure since at least the 1927 edition of this Code.

4.4.2. Unless otherwise noted, any additions, deletions or Revisions that are indicated in responses to RFIs or on the approved Shop and Erection Drawings shall constitute authorization by the Owner that the additions, deletions or revisions are Released for Construction. The Fabricator and the Erector shall promptly notify the Owner's Designated Representative for Construction when any direction or notation in responses to RFIs or on the Shop or Erection Drawings or other information will result in an additional cost and/or a delay. See Sections 3.5 and 9.3.

Commentary:
When the Fabricator notifies the Owner's Designated Representative for Construction that a direction or notation in responses to RFIs or on the Shop or Erection Drawings will result in an additional cost or a delay, it is then normally the responsibility of the Owner's Designated Representative for Construction to subsequently notify the Owner's Designated Representative for Design.

4.5. Shop and/or Erection Drawings Not Furnished by the Fabricator

When the Shop and Erection Drawings are not prepared by the Fabricator, but are furnished by others, they shall be delivered to the Fabricator in a timely manner. These Shop and Erection Drawings shall be prepared, insofar as is practical, in accordance with the shop fabrication and detailing standards of the Fabricator. The Fabricator shall neither be responsible for the completeness or accuracy of Shop and Erection Drawings so furnished, nor for the general fit-up of the members that are fabricated from them.

4.6. The RFI Process

When Requests for Information (RFIs) are issued, the process shall include the maintenance of a written record of inquiries and responses related to interpretation and implementation of the Contract Documents, including the Clarifications and/or Revisions to the Contract Documents that result, if any. RFIs shall not be used for the incremental Release for Construction of Design Drawings. When RFIs involve discrepancies or Revisions, see Sections 3.3, 3.5, and 4.4.2.

Commentary:

The RFI process is most commonly used during the detailing process, but can also be used to forward inquiries by the Erector or to inform the Owners Designated Representative For Design in the event of a fabricator or erector error and to develop corrective measures to resolve such errors.

The RFI process is intended to provide a written record of inquiries and associated responses but not to replace all verbal communication between the parties on the project. RFIs should be prepared and responded to in a timely fashion so as not to delay the work of the Detailer, Fabricator, and Erector. Discussion of the RFI issues and possible solutions between the Fabricator, Erector, and Owner's Designated Representatives for Design and Construction often can facilitate timely and practical resolution. Unlike Shop and Erection Drawing submittals in Section 4.2, RFI response time can vary depending on the urgency of the issue, the amount of work required by the Owner's Designated Representatives for Design and Construction to develop a complete response, and other circumstances such as building official approval.

RFIs should be prepared in a standardized format, including RFI number and date, identity of the author, reference to a specific Design Drawing number (and specific detail as applicable) or Specification section, the needed response date, a description of a suggested solution (graphic depictions are recommended for more complex issues), and an indication of possible schedule and cost impacts. RFIs should be limited to one question each (unless multiple questions are interrelated to the same issue) to facilitate the resolution and minimize response time. Questions and proposed solutions presented in RFIs should be clear and complete. RFI responses should be equally clear and complete in the depictions of the solutions, and signed and dated by the responding party.

Unless otherwise noted, the Fabricator/Erector can assume that a response to an RFI constitutes a Release for Construction. However, if the response will result in an increase in cost or a delay in schedule, Section 4.4.2 requires that the Fabricator/Erector promptly inform the Owner's Designated Representatives for Design and Construction.

SECTION 5. MATERIALS

5.1. Mill Materials

Unless otherwise noted in the Contract Documents, the Fabricator is permitted to order the materials that are necessary for fabrication when the Fabricator receives Contract Documents that have been Released for Construction.

Commentary:
The Fabricator may purchase materials in stock lengths, exact lengths or multiples of exact lengths to suit the dimensions shown in the structural Design Drawings. Such purchases will normally be job-specific in nature and may not suitable for use on other projects or returned for full credit if subsequent design changes make these materials unsuitable for their originally intended use. The Fabricator should be paid for these materials upon delivery from the mill, subject to appropriate additional payment or credit if subsequent unanticipated modification or reorder is required. Purchasing materials to exact lengths is not considered fabrication.

5.1.1. Unless otherwise specified by means of special testing requirements in the Contract Documents, mill testing shall be limited to those tests that are required for the material in the ASTM specifications indicated in the Contract Documents. Materials ordered to special material requirements shall be marked by the supplier as specified in ASTM A6/A6M Section 12 prior to delivery to the Fabricator's shop or other point of use. Such material not so marked by the supplier, shall not be used until:

(a) Its identification is established by means of testing in accordance with the applicable ASTM specifications; and,
(b) A Fabricator's identification mark, as described in Section 6.1.2 and 6.1.3, has been applied.

5.1.2. When Mill Material does not satisfy ASTM A6/A6M tolerances for camber, profile, flatness or sweep, the Fabricator shall be permitted to perform corrective procedures, including the use of controlled heating and/or mechanical straightening, subject to the limitations in the AISC Specification.

Commentary:
Mill dimensional tolerances are completely set forth in ASTM A6/A6M. Normal variations in the cross-sectional geometry of Standard Structural Shapes must be recognized by the designer, the Fabricator, the Steel Detailer and the Erector (for example, see Figure C–5.1). Such tolerances are mandatory because roll wear, thermal distortions of the hot cross-section immediately after leaving the forming rolls and differential cooling distortions that take place on the cooling beds are all unavoidable. Geometric perfection of the cross-section is

not necessary for either structural or architectural reasons, if the tolerances are recognized and provided for.

ASTM A6/A6M also stipulates tolerances for straightness that are adequate for typical construction. However, these characteristics may be controlled or corrected to closer tolerances during the fabrication process when the added cost is justified by the special requirements for an atypical project.

5.1.3. When variations that exceed ASTM A6/A6M tolerances are discovered or occur after the receipt of Mill Material the Fabricator shall, at the Fabricator's option, be permitted to perform the ASTM A6/A6M corrective procedures for mill reconditioning of the surface of Structural Steel shapes and plates.

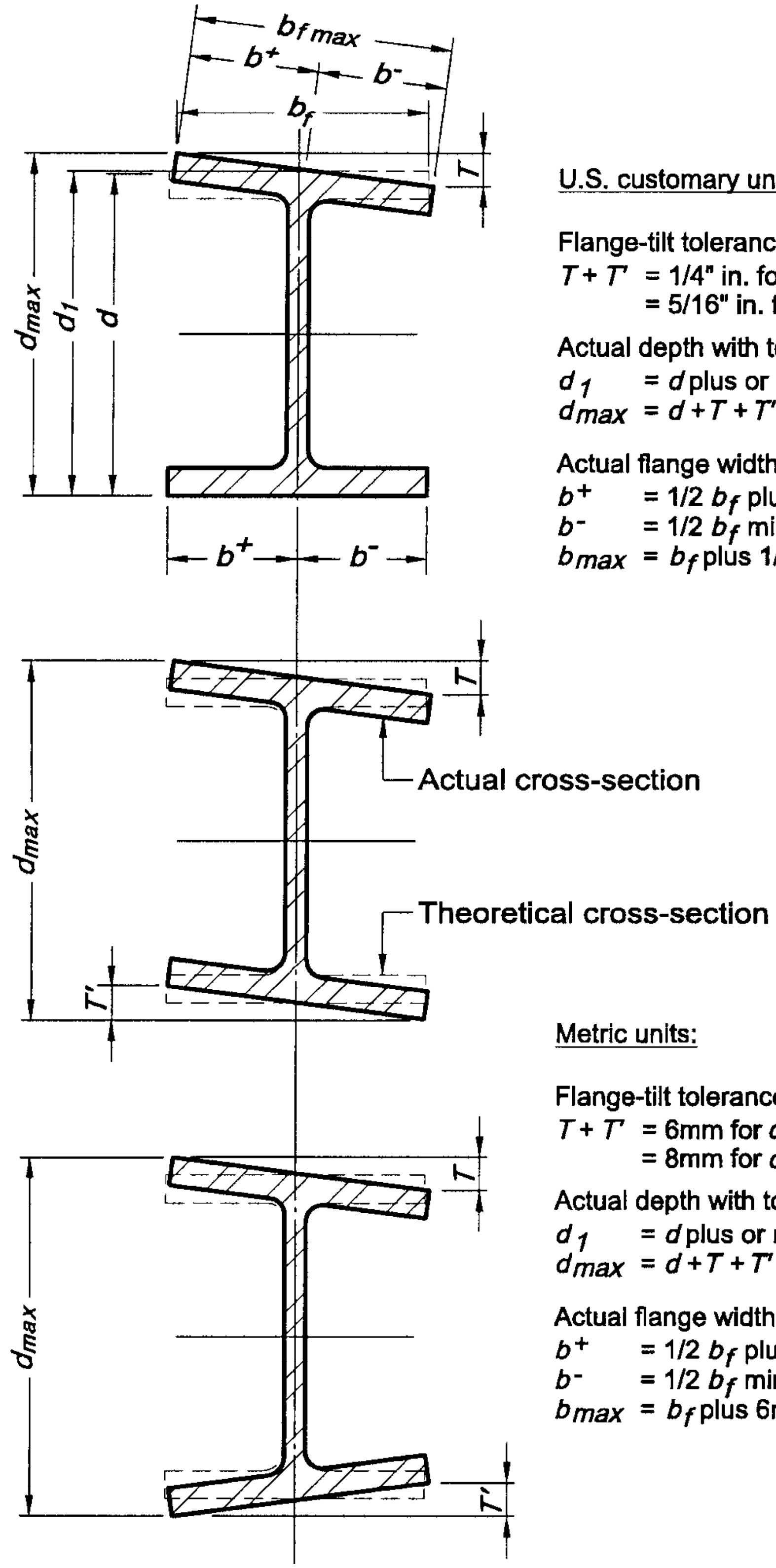
5.1.4. When special tolerances that are more restrictive than those in ASTM A6/A6M are required for Mill Materials, such special tolerances shall be specified in the Contract Documents. The Fabricator shall, at the Fabricator's option, be permitted to order material to ASTM A6/A6M tolerances and subsequently perform the corrective procedures described in Sections 5.1.2 and 5.1.3.

5.2. Stock Materials

5.2.1. If used for structural purposes, materials that are taken from stock by the Fabricator shall be of a quality that is at least equal to that required in the ASTM specifications indicated in the Contract Documents.

5.2.2. Certified mill test reports shall be accepted as sufficient record of the quality of materials taken from stock by the Fabricator. The Fabricator shall review and retain the certified mill test reports that cover such stock materials. However, the Fabricator need not maintain records that identify individual pieces of stock material against individual certified mill test reports, provided the Fabricator purchases stock materials that meet the requirements for material grade and quality in the applicable ASTM specifications.

5.2.3. Stock materials that are purchased under no particular specification, under a specification that is less rigorous than the applicable ASTM specifications or without certified mill test reports or other recognized test reports shall not be used without the approval of the Owner's Designated Representative for Design.



U.S. customary units:

Flange-tilt tolerances:
$T + T'$ = 1/4" in. for $d \leq 12$ in.
= 5/16" in. for $d > 12$ in.

Actual depth with tolerances:
d_1 = d plus or minus 1/8 in. (typ.)
d_{max} = $d + T + T'$

Actual flange width with tolerances:
b^+ = 1/2 b_f plus or minus 3/16 in.
b^- = 1/2 b_f minus or plus 3/16 in.
b_{max} = b_f plus 1/4 in. or minus 3/16 in.

Metric units:

Flange-tilt tolerances:
$T + T'$ = 6mm for $d \leq$ 300mm
= 8mm for $d >$ 300mm

Actual depth with tolerances:
d_1 = d plus or minus 3mm
d_{max} = $d + T + T'$

Actual flange width with tolerances:
b^+ = 1/2 b_f plus or minus 5mm
b^- = 1/2 b_f minus or plus 5mm
b_{max} = b_f plus 6mm or minus 5mm

Figure C-5.1. Mill tolerances on the cross-section of a W-shape.

SECTION 6. SHOP FABRICATION AND DELIVERY

6.1. Identification of Material

6.1.1. The Fabricator shall be able to demonstrate by written procedure and actual practice a method of material identification, visible up to the point of assembling members as follows:

(a) For shop-standard material, identification capability shall include shape designation. Representative mill test reports shall be furnished by the Fabricator if requested to do so by the Owner's Designated Representative for Design, either in the Contract Documents or in separate written instructions given to the Fabricator prior to ordering Mill Materials.
(b) For material of grade other than shop-standard material, identification capability shall include shape designation and material grade. Representative mill test reports shall be furnished by the Fabricator if requested to do so by the Owner's Designated Representative for Design, either in the Contract Documents or in separate written instructions given to the Fabricator prior to ordering Mill Materials.
(c) For material ordered in accordance with an ASTM supplement or other special material requirements in the Contract Documents, identification capability shall include shape designation, material grade, and heat number. The corresponding mill test reports shall be furnished by the Fabricator if requested to do so by the Owner's Designated Representative for Design, either in the Contract Documents or in separate written instructions given to the Fabricator prior to ordering Mill Materials.

Unless an alternative system is established in the Fabricator's written procedures, shop-standard material shall be as follows:

Material	**Shop-standard material grade**
W and WT	ASTM A992
M, S, MT and ST	ASTM A36
HP	ASTM A36
L	ASTM A36
C and MC	ASTM A36
HSS	ASTM A500 grade B
Steel Pipe	ASTM A53 grade B
Plates and Bars	ASTM A36

Commentary:
The requirements in Section 6.1.1(a) will suffice for most projects. When material is of a strength level that differs from the shop-standard grade, the requirements in Section 6.1.1(b) apply. When special material requirements

apply, such as ASTM A6/A6M supplement S5 or S30 for CVN testing, ASTM A6/A6M supplement S8 for ultrasonic testing, or ASTM A588/A588M for atmospheric corrosion resistance, the requirements in Section 6.1.1(c) are applicable.

6.1.2. During fabrication, up to the point of assembling members, each piece of material that is ordered to special material requirements shall carry a Fabricator's identification mark or an original supplier's identification mark. The Fabricator's identification mark shall be in accordance with the Fabricator's established material identification system, which shall be on record and available prior to the start of fabrication for the information of the Owner's Designated Representative for Construction, the building-code authority and the Inspector.

6.1.3. Members that are made of material that is ordered to special material requirements shall not be given the same assembling or erection mark as members made of other material, even if they are of identical dimensions and detail.

6.2. Preparation of Material

6.2.1. The thermal cutting of Structural Steel by hand-guided or mechanically guided means is permitted.

6.2.2. Surfaces that are specified as "finished" in the Contract Documents shall have a roughness height value measured in accordance with ANSI/ASME B46.1 that is equal to or less than 500. The use of any fabricating technique that produces such a finish is permitted.

Commentary:
Most cutting processes, including friction sawing and cold sawing, and milling processes meet a surface roughness limitation of 500 per ANSI/ASME B46.1.

6.3. Fitting and Fastening

6.3.1. Projecting elements of Connection materials need not be straightened in the connecting plane, subject to the limitations in the AISC Specification.

6.3.2. Backing bars and runoff tabs shall be used in accordance with AWS D1.1 as required to produce sound welds. The Fabricator or Erector need not remove backing bars or runoff tabs unless such removal is specified in the Contract Documents. When the removal of backing bars is specified in the Contract Documents, such removal shall meet the requirements in AWS D1.1. When the removal of runoff tabs is specified in the Contract Documents, hand flame-cutting close to the edge of the finished member with no further finishing is permitted, unless other finishing is specified in the Contract Documents.

Commentary:
In most cases, the treatment of backing bars and runoff tabs is left to the discretion of the Owner's Designated Representative for Design. In some cases, treatment beyond the basic cases described in this Section may be required. As one example, special treatment is required for backing bars and runoff tabs in beam-to-column moment Connections when the requirements in the AISC Seismic Provisions must be met. In all cases, the Owner's Designated Representative for Design should specify the required treatments in the Contract Documents.

6.3.3. Unless otherwise noted in the Shop Drawings, high-strength bolts for shop-attached Connection material shall be installed in the shop in accordance with the requirements in the AISC Specification.

6.4. Fabrication Tolerances

The tolerances on Structural Steel fabrication shall be in accordance with the requirements in Section 6.4.1 through 6.4.6.

Commentary:
Fabrication tolerances are stipulated in several specifications and codes, each applicable to a specialized area of construction. Basic fabrication tolerances are stipulated in this Section. For Architecturally Exposed Structural Steel, see Section 10. Other specifications and codes are also commonly incorporated by reference in the Contract Documents, such as the AISC Specification, the RCSC Specification, AWS D1.1 and the AASHTO Specification.

6.4.1. For members that have both ends finished (see Section 6.2.2) for contact bearing, the variation in the overall length shall be equal to or less than 1/32 in. [1 mm]. For other members that frame to other Structural Steel elements, the variation in the detailed length shall be as follows:

(a) For members that are equal to or less than 30 ft [9 000 mm] in length, the variation shall be equal to or less than 1/16 in. [2 mm].
(b) For members that are greater than 30 ft [9 000 mm] in length, the variation shall be equal to or less than 1/8 in. [3 mm].

6.4.2. For straight structural members other than compression members, whether of a single Standard Structural Shape or built-up, the variation in straightness shall be equal to or less than that specified for wide-flange shapes in ASTM A6/A6M, except when a smaller variation in straightness is specified in the Contract Documents. For straight compression members, whether of a Standard

Structural Shape or built-up, the variation in straightness shall be equal to or less than 1/1000 of the axial length between points that are to be laterally supported. For curved structural members, the variation from the theoretical curvature shall be equal to or less than the variation in sweep that is specified for an equivalent straight member of the same straight length in ASTM A6/A6M.

In all cases, completed members shall be free of twists, bends and open joints. Sharp kinks or bends shall be cause for rejection.

6.4.3. For beams and trusses that are detailed without specified camber, the member shall be fabricated so that, after erection, any incidental camber due to rolling or shop fabrication is upward.

6.4.4. For beams that are specified in the Contract Documents with camber, beams received by the Fabricator with 75% of the specified camber shall require no further cambering. Otherwise, the variation in camber shall be as follows:

(a) For beams that are equal to or less than 50 ft [15 000 mm] in length, the variation shall be equal to or less than minus zero / plus 1/2 in. [13 mm].
(b) For beams that are greater than 50 ft [15 000 mm] in length, the variation shall be equal to or less than minus zero / plus 1/2 in. plus 1/8 in. for each 10 ft or fraction thereof [13 mm plus 3 mm for each 3 000 mm or fraction thereof] in excess of 50 ft [15 000 mm] in length.

For the purpose of inspection, camber shall be measured in the Fabricator's shop in the unstressed condition.

Commentary:
There is no known way to inspect beam camber after the beam is received in the field because of factors that include:

(a) The release of stresses in members over time and in varying applications;
(b) The effects of the dead weight of the member;
(c) The restraint caused by the end Connections in the erected state; and,
(d) The effects of additional dead load that may ultimately be intended to be applied, if any.

Therefore, inspection of the Fabricator's work on beam camber must be done in the fabrication shop in the unstressed condition.

6.4.5. For fabricated trusses that are specified in the Contract Documents with camber, the variation in camber at each specified camber point shall be equal to or less than plus or minus 1/800 of the distance to that point from the nearest point of support. For the purpose of inspection, camber shall be measured in the Fabricator's shop in the unstressed condition. For fabricated trusses that are specified in the Contract Documents without indication of camber, the foregoing requirements shall be applied at each panel point of the truss with a zero camber ordinate.

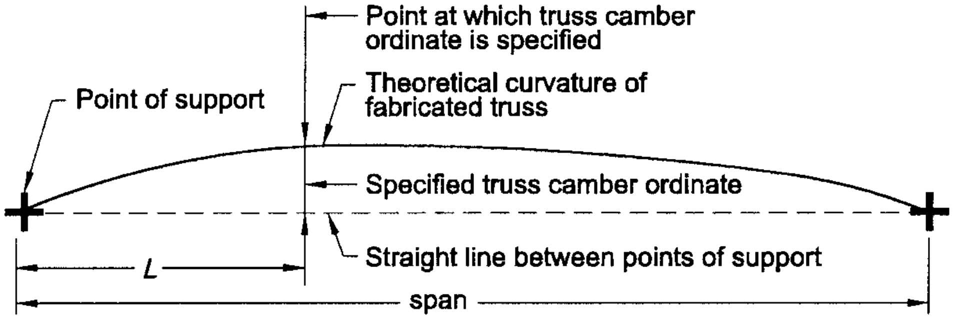
Commentary:
There is no known way to inspect truss camber after the truss is received in the field because of factors that include:

(a) The effects of the dead weight of the member;
(b) The restraint caused by the truss Connections in the erected state; and,
(c) The effects of additional dead load that may ultimately be intended to be applied, if any.

Therefore, inspection of the Fabricator's work on truss camber must be done in the fabrication shop in the unstressed condition. See Figure C–6.1.

6.4.6. When permissible variations in the depths of beams and girders result in abrupt changes in depth at splices, such deviations shall be accounted for as follows:

(a) For splices with bolted joints, the variations in depth shall be taken up with filler plates; and,
(b) For splices with welded joints, the weld profile shall be adjusted to conform to the variations in depth, the required cross-section of weld shall be provided and the slope of the weld surface shall meet the requirements in AWS D1.1.



Taking *L* as the distance from the point at which truss camber is specified to the closer point of support, in. [mm], the tolerance on truss camber at that point is calculated as *L*/800. *L* must be equal to or less than one-half the span.

Figure C-6.1. Illustration of the tolerance on camber for fabricated trusses with specified camber.

6.5. Shop Cleaning and Painting (see also Section 3.1.6)

Structural Steel that does not require shop paint shall be cleaned of oil and grease with solvent cleaners, and of dirt and other foreign material by sweeping with a fiber brush or other suitable means. For Structural Steel that is required to be shop painted, the requirements in Sections 6.5.1 through 6.5.4 shall apply.

Commentary:

Extended exposure of unpainted Structural Steel that has been cleaned for the subsequent application of fire protection materials can be detrimental to the fabricated product. Most levels of cleaning require the removal of all loose mill scale, but permit some amount of tightly adhering mill scale. When a piece of Structural Steel that has been cleaned to an acceptable level is left exposed to a normal environment, moisture can penetrate behind the scale, and some “lifting” of the scale by the oxidation process is to be expected. Cleanup of “lifted” mill scale is not the responsibility of the Fabricator, but is to be assigned by contract requirement to an appropriate contractor.

Section 6.5.4 of this Code is not applicable to weathering steel, for which special cleaning specifications are always required in the Contract Documents.

6.5.1. The Fabricator is not responsible for deterioration of the shop coat that may result from exposure to ordinary atmospheric conditions or corrosive conditions that are more severe than ordinary atmospheric conditions.

Commentary:

The shop coat of paint is the prime coat of the protective system. It is intended as protection for only a short period of exposure in ordinary atmospheric conditions, and is considered a temporary and provisional coating.

6.5.2. Unless otherwise specified in the Contract Documents, the Fabricator shall, as a minimum, hand clean the Structural Steel of loose rust, loose mill scale, dirt and other foreign matter, prior to painting, by means of wire brushing or by other methods elected by the Fabricator, to meet the requirements of SSPC-SP2. If the Fabricator’s workmanship on surface preparation is to be inspected by the Inspector, such inspection shall be performed in a timely manner prior to the application of the shop coat.

Commentary:

The selection of a paint system is a design decision involving many factors including:

(a) The Owner’s preference;

(b) The service life of the structure;
(c) The severity of environmental exposure;
(d) The cost of both initial application and future renewals; and,
(e) The compatibility of the various components that comprise the paint system (surface preparation, shop coat and subsequent coats).

Because the inspection of shop painting must be concerned with workmanship at each stage of the operation, the Fabricator provides notice of the schedule of operations and affords the Inspector access to the work site. Inspection must then be coordinated with that schedule so as to avoid delay of the scheduled operations.

Acceptance of the prepared surface must be made prior to the application of the shop coat because the degree of surface preparation cannot be readily verified after painting. Time delay between surface preparation and the application of the shop coat can result in unacceptable deterioration of a properly prepared surface, necessitating a repetition of surface preparation. This is especially true with blast-cleaned surfaces. Therefore, to avoid potential deterioration of the surface, it is assumed that surface preparation is accepted unless it is inspected and rejected prior to the scheduled application of the shop coat.

The shop coat in any paint system is designed to maximize the wetting and adherence characteristics of the paint, usually at the expense of its weathering capabilities. Deterioration of the shop coat normally begins immediately after exposure to the elements and worsens as the duration of exposure is extended. Consequently, extended exposure of the shop coat will likely lead to its deterioration and may necessitate repair, possibly including the repetition of surface preparation and shop coat application in limited areas. With the introduction of high-performance paint systems, avoiding delay in the application of the shop coat has become more critical. High-performance paint systems generally require a greater degree of surface preparation, as well as early application of weathering protection for the shop coat.

Since the Fabricator does not control the selection of the paint system, the compatibility of the various components of the total paint system, or the length of exposure of the shop coat, the Fabricator cannot guarantee the performance of the shop coat or any other part of the system. Instead, the Fabricator is responsible only for accomplishing the specified surface preparation and for applying the shop coat (or coats) in accordance with the Contract Documents.

This Section stipulates that the Structural Steel is to be cleaned to meet the requirements in SSPC-SP2. This stipulation is not intended to represent an exclusive cleaning level, but rather the level of surface preparation that will be furnished unless otherwise specified in the Contract Documents if the Structural Steel is to be painted.

Further information regarding shop painting is available in *A Guide to Shop Painting of Structural Steel*, published jointly by SSPC and AISC.

6.5.3. Unless otherwise specified in the Contract Documents, paint shall be applied by brushing, spraying, rolling, flow coating, dipping or other suitable means, at the election of the Fabricator. When the term “shop coat”, “shop paint” or other equivalent term is used with no paint system specified, the Fabricator’s standard shop paint shall be applied to a minimum dry-film thickness of one mil [25 μm].

6.5.4. Touch-up of abrasions caused by handling after painting shall be the responsibility of the contractor that performs touch-up in the field or field painting.

Commentary:
Touch-up in the field and field painting are not normally part of the Fabricator’s or the Erector’s contract.

6.6. Marking and Shipping of Materials

6.6.1. Unless otherwise specified in the Contract Documents, erection marks shall be applied to the Structural Steel members by painting or other suitable means.

6.6.2. Bolt assemblies and loose bolts, nuts and washers shall be shipped in separate closed containers according to length and diameter, as applicable. Pins and other small parts and packages of bolts, nuts and washers shall be shipped in boxes, crates, kegs or barrels. A list and description of the material shall appear on the outside of each closed container.

Commentary:
In most cases bolts, nuts and other components in a fastener assembly can be shipped loose in separate containers. However, ASTM F1852/F1852M twist-off-type tension-control bolt assemblies and galvanized ASTM A325, A325M and F1852/F1852M bolt assemblies must be assembled and shipped in the same container according to length and diameter.

6.7. Delivery of Materials

6.7.1. Fabricated Structural Steel shall be delivered in a sequence that will permit efficient and economical fabrication and erection, and that is consistent with requirements in the Contract Documents. If the Owner or Owner’s Designated Representative for Construction wishes to prescribe or control the sequence of delivery of materials, that entity shall specify the required sequence in the Contract Documents. If the Owner’s Designated Representative for Construction contracts separately for delivery and for erection, the Owner’s

Designated Representative for Construction shall coordinate planning between contractors.

6.7.2. Anchor Rods, washers, nuts and other anchorage or grillage materials that are to be built into concrete or masonry shall be shipped so that they will be available when needed. The Owner's Designated Representative for Construction shall allow the Fabricator sufficient time to fabricate and ship such materials before they are needed.

6.7.3. If any shortage is claimed relative to the quantities of materials that are shown in the shipping statements, the Owner's Designated Representative for Construction or the Erector shall promptly notify the Fabricator so that the claim can be investigated.

Commentary:
The quantities of material that are shown in the shipping statement are customarily accepted as correct by the Owner's Designated Representative for Construction, the Fabricator and the Erector.

6.7.4. Unless otherwise specified in the Contract Documents, and subject to the approved Shop and Erection Drawings, the Fabricator shall limit the number of field splices to that consistent with minimum project cost.

Commentary:
This Section recognizes that the size and weight of Structural Steel assemblies may be limited by shop capabilities, the permissible weight and clearance dimensions of available transportation or job-site conditions.

6.7.5. If material arrives at its destination in damaged condition, the receiving entity shall promptly notify the Fabricator and carrier prior to unloading the material, or promptly upon discovery prior to erection.

SECTION 7. ERECTION

7.1. Method of Erection

Fabricated Structural Steel shall be erected using methods and a sequence that will permit efficient and economical performance of erection, and that is consistent with the requirements in the Contract Documents. If the Owner or Owner's Designated Representative for Construction wishes to prescribe or control the method and/or sequence of erection, or specifies that certain members cannot be erected in their normal sequence, that entity shall specify the required method and sequence in the Contract Documents. If the Owner's Designated Representative for Construction contracts separately for fabrication services and for erection services, the Owner's Designated Representative for Construction shall coordinate planning between contractors.

Commentary:
Design modifications are sometimes requested by the Erector to allow or facilitate the erection of the Structural Steel frame. When this is the case, the Erector should notify the Fabricator prior to the preparation of Shop and Erection Drawings so that the Fabricator may refer the Erector's request to the Owner's Designated Representatives for Design and Construction for resolution.

7.2. Job-Site Conditions

The Owner's Designated Representative for Construction shall provide and maintain the following for the Fabricator and the Erector:

(a) Adequate access roads into and through the job site for the safe delivery and movement of the material to be erected and of derricks, cranes, trucks and other necessary equipment under their own power;
(b) A firm, properly graded, drained, convenient and adequate space at the job site for the operation of the Erector's equipment, free from overhead obstructions, such as power lines, telephone lines or similar conditions; and,
(c) Adequate storage space, when the structure does not occupy the full available job site, to enable the Fabricator and the Erector to operate at maximum practical speed.

Otherwise, the Owner's Designated Representative for Construction shall inform the Fabricator and the Erector of the actual job-site conditions and/or special delivery requirements prior to bidding.

7.3. Foundations, Piers and Abutments

The accurate location, strength and suitability of, and access to, all foundations, piers and abutments shall be the responsibility of the Owner's Designated Representative for Construction.

7.4. Lines and Bench Marks

The Owner's Designated Representative for Construction shall be responsible for the accurate location of lines and benchmarks at the job site and shall furnish the Erector with a plan that contains all such information. The Owner's Designated Representative for Construction shall establish offset lines and reference elevations at each level for the Erector's use in the positioning of Adjustable Items (see Section 7.13.1.3), if any.

7.5. Installation of Anchor Rods, Foundation Bolts and Other Embedded Items

7.5.1. Anchor Rods, foundation bolts and other embedded items shall be set by the Owner's Designated Representative for Construction in accordance with Embedment Drawings that have been approved by the Owner's Designated Representatives for Design and Construction. The variation in location of these items from the dimensions shown in the Embedment Drawings shall be as follows:

(a) The variation in dimension between the centers of any two Anchor Rods within an Anchor-Rod Group shall be equal to or less than 1/8 in. [3 mm].
(b) The variation in dimension between the centers of adjacent Anchor-Rod Groups shall be equal to or less than 1/4 in. [6 mm].
(c) The variation in elevation of the tops of Anchor Rods shall be equal to or less than plus or minus 1/2 in. [13 mm].
(d) The accumulated variation in dimension between centers of Anchor-Rod Groups along the Column Line through multiple Anchor-Rod Groups shall be equal to or less than 1/4 in. per 100 ft [2 mm per 10 000 mm], but not to exceed a total of 1 in. [25 mm].
(e) The variation in dimension from the center of any Anchor-Rod Group to the Column Line through that group shall be equal to or less than 1/4 in. [6 mm].

The tolerances that are specified in (b), (c) and (d) shall apply to offset dimensions shown in the structural Design Drawings, measured parallel and perpendicular to the nearest Column Line, for individual columns that are shown in the structural Design Drawings as offset from Column Lines.

Commentary:
The tolerances established in this Section have been selected for compatibility with the holes sizes that are recommended for base plates in the AISC Manual of Steel Construction. If special conditions require more restrictive tolerances, the contractor responsible for setting the Anchor Rods should be so informed in the Contract Documents. When the Anchor Rods are set in sleeves, the

adjustment provided may be used to satisfy the required Anchor-Rod setting tolerances.

7.5.2. Unless otherwise specified in the Contract Documents, Anchor Rods shall be set with their longitudinal axis perpendicular to the theoretical bearing surface.

7.5.3. Embedded items and Connection materials that are part of the work of other trades, but that will receive Structural Steel, shall be located and set by the Owner's Designated Representative for Construction in accordance with an approved Embedment Drawing. The variation in location of these items shall be limited to a magnitude that is consistent with the tolerances that are specified in Section 7.13 for the erection of the Structural Steel.

7.5.4. All work that is performed by the Owner's Designated Representative for Construction shall be completed so as not to delay or interfere with the work of the Fabricator and the Erector. The Owner's Designated Representative for Construction shall conduct a survey of the as-built locations of Anchor Rods, foundation bolts and other embedded items, and shall verify that all items covered in Section 7.5 meet the corresponding tolerances. When corrective action is necessary, the Owner's Designated Representative for Construction shall obtain the guidance and approval of the Owner's Designated Representative for Design.

Commentary:
Few Fabricators or Erectors have the capability to provide this survey. Under standard practice, it is the responsibility of others.

7.6. Installation of Bearing Devices

All leveling plates, leveling nuts and washers and loose base and bearing plates that can be handled without a derrick or crane are set to line and grade by the Owner's Designated Representative for Construction. Loose base and bearing plates that require handling with a derrick or crane shall be set by the Erector to lines and grades established by the Owner's Designated Representative for Construction. The Fabricator shall clearly scribe loose base and bearing plates with lines or other suitable marks to facilitate proper alignment.

Promptly after the setting of Bearing Devices, the Owner's Designated Representative for Construction shall check them for line and grade. The variation in elevation relative to the established grade for all Bearing Devices shall be equal to or less than plus or minus 1/8 in. [3 mm]. The final location of Bearing Devices shall be the responsibility of the Owner's Designated Representative for Construction.

Commentary:
The 1/8 in. [3 mm] tolerance on elevation of Bearing Devices relative to established grades is provided to permit some variation in setting Bearing Devices, and to account for the accuracy that is attainable with standard surveying instruments. The use of leveling plates larger than 22 in. by 22 in. [550 mm by 550 mm] is discouraged and grouting is recommended with larger sizes. For the purposes of erection stability, the use of leveling nuts and washers is discouraged when base plates have less than four Anchor Rods.

7.7. Grouting

Grouting shall be the responsibility of the Owner's Designated Representative for Construction. Leveling plates and loose base and bearing plates shall be promptly grouted after they are set and checked for line and grade. Columns with attached base plates, beams with attached bearing plates and other similar members with attached Bearing Devices that are temporarily supported on leveling nuts and washers, shims or other similar leveling devices, shall be promptly grouted after the Structural Steel frame or portion thereof has been plumbed.

Commentary:
In the majority of structures the vertical load from the column bases is transmitted to the foundations through structural grout. In general, there are three methods by which support is provided for column bases during erection:

(a) Pre-grouted leveling plates or loose base plates;
(b) Shims; and,
(c) Leveling nuts and washers on the Anchor Rods beneath the column base.

Standard practice provides that loose base plates and leveling plates are to be grouted as they are set. Bearing Devices that are set on shims or leveling nuts are grouted after plumbing, which means that the weight of the erected Structural Steel frame is supported on the shims or washers, nuts and Anchor Rods. The Erector must take care to ensure that the load that is transmitted in this temporary condition does not exceed the strength of the shims or washers, nuts and Anchor Rods. These considerations are presented in greater detail in AISC Design Guides No. 1 and 10.

7.8. Field Connection Material

7.8.1. The Fabricator shall provide field Connection details that are consistent with the requirements in the Contract Documents and that will, in the Fabricator's opinion, result in economical fabrication and erection.

7.8.2. When the Fabricator is responsible for erecting the Structural Steel, the Fabricator shall furnish all materials that are required for both temporary and permanent Connection of the component parts of the Structural Steel frame.

7.8.3. When the erection of the Structural Steel is not performed by the Fabricator, the Fabricator shall furnish the following field Connection material:

(a) Bolts, nuts and washers of the required grade, type and size and in sufficient quantity for all Structural Steel-to-Structural Steel field Connections that are to be permanently bolted, including an extra 2 percent of each bolt size (diameter and length);
(b) Shims that are shown as necessary for make-up of permanent Structural Steel-to-Structural Steel Connections; and,
(c) Backing bars and run-off tabs that are required for field welding.

7.8.4. The Erector shall furnish all welding electrodes, fit-up bolts and drift pins used for the erection of the Structural Steel.

Commentary:
See the commentary for Section 2.2.

7.9. Loose Material

Unless otherwise specified in the Contract Documents, loose Structural Steel items that are not connected to the Structural Steel frame shall be set by the Owner's Designated Representative for Construction without assistance from the Erector.

7.10. Temporary Support of Structural Steel Frames

7.10.1. The Owner's Designated Representative for Design shall identify the following in the Contract Documents:

(a) The lateral-load-resisting system and connecting diaphragm elements that provide for lateral strength and stability in the completed structure; and,
(b) Any special erection conditions or other considerations that are required by the design concept, such as the use of shores, jacks or loads that must be adjusted as erection progresses to set or maintain camber, position within specified tolerances or prestress.

Commentary:
The intent of Section 7.10.1 of the Code is to alert the Owners Designated Representative for Construction and the Erector of the means for lateral load resistance in the completed structure so that appropriate planning can occur for

construction of the building. Examples of a description of the lateral load resisting system as required by 7.10.1(a) are shown below.

Example 1 is an all-steel building with a composite metal deck and concrete floor system. All lateral load resistance is provided by welded moment frames in each orthogonal building direction. One suitable description of this lateral load resisting system is:

All lateral load resistance and stability of the building in the completed structure is provided by moment frames with welded beam to column connections framed in each orthogonal direction (see plan sheets for locations). The composite metal deck and concrete floors serve as horizontal diaphragms that distribute the lateral wind and seismic forces horizontally to the vertical moment frames. The vertical moment frames carry the applied lateral loads to the building foundation.

Example 2 is a steel-framed building with a composite metal deck and concrete floor system. All beam-to-column connections are simple connections and all lateral load resistance is provided by reinforced concrete shear walls in the building core and in the stair wells. One suitable description of this lateral load resisting system is:

All lateral load resistance and stability of the building in the completed structure is provided exclusively by cast-in-place reinforced concrete shear walls in the building core and stair wells (see plan sheets for locations). These walls provide all lateral load resistance in each orthogonal building direction. The composite metal deck and concrete floors serve as horizontal diaphragms that distribute the lateral wind and seismic forces horizontally to the concrete shear walls. The concrete shear walls carry the applied lateral loads to the building foundation.

See also Commentary Section 7.10.3.

7.10.2. The Owner's Designated Representative for Construction shall indicate to the Erector prior to bidding, the installation schedule for non-Structural Steel elements of the lateral-load-resisting system and connecting diaphragm elements identified by the Owner's Designated Representative for Design in the Contract Documents.

Commentary:
See Commentary Section 7.10.3.

7.10.3. Based upon the information provided in accordance with Sections 7.10.1 and 7.10.2, the Erector shall determine, furnish and install all temporary supports, such as temporary guys, beams, falsework, cribbing or other elements required for the erection operation. These temporary supports shall be sufficient to secure the bare Structural Steel framing or any portion thereof against loads that are likely to be encountered during erection, including those due to wind and those that result from erection operations.

The Erector need not consider loads during erection that result from the performance of work by, or the acts of, others, except as specifically identified by the Owner's Designated Representatives for Design and Construction, nor those that are unpredictable, such as loads due to hurricane, tornado, earthquake, explosion or collision.

Temporary supports that are required during or after the erection of the Structural Steel frame for the support of loads caused by non-Structural Steel elements, including cladding, interior partitions and other such elements that will induce or transmit loads to the Structural Steel frame during or after erection, shall be the responsibility of others.

Commentary:

Many Structural Steel frames have lateral-load-resisting systems that are activated during the erection process. Such lateral-load-resisting systems may consist of welded moment frames, braced frames or, in some instances, columns that cantilever from fixed-base foundations. Such frames are normally braced with temporary guys that, together with the steel deck floor and roof diaphragms, or other diaphragm bracing that may be included as part of the design, provide stability during the erection process. The guy cables are also commonly used to plumb the Structural Steel frame. The Erector normally furnishes and installs the required temporary supports and bracing to secure the bare Structural Steel frame, or portion thereof, during the erection process. When Erection Bracing Drawings are required in the Contract Documents, those drawings show this information.

If the Owner's Designated Representative for Construction determines that steel decking is not installed by the Erector, temporary diaphragm bracing may be required if a horizontal diaphragm is not available to distribute loads to the vertical and lateral load resisting system. If the steel deck will not be available as a diaphragm during Structural Steel erection, the Owner's Designated Representative for Construction must communicate this condition to the Erector prior to bidding. If such diaphragm bracing is required, it must be furnished and installed by the Erector.

Sometimes structural systems that are employed by the Owner's Designated Representative for Design rely upon other elements besides the Structural Steel frame for lateral-load resistance. For instance, concrete or masonry shear walls or precast spandrels may be used to provide resistance to vertical and lateral loads in the completed structure. Because these situations may not be obvious to the contractor or the Erector, it is required in this Code that the Owner's Designated Representative for Design identify such situations in the Contract Documents. Similarly, if a structure is designed so that special erection techniques are required, such as jacking to impose certain loads or position during erection, it is required in this Code that such requirements be specifically identified in the Contract Documents.

In some instances, the Owner's Designated Representative for Design may elect to show erection bracing in the Design Drawings. When this is the case, the Owner's Designated Representative for Design should then confirm that the bracing requirements were understood by review and approval of the Erection Drawings during the submittal process.

Sometimes during construction of a building, collateral building elements, such as exterior cladding, may be required to be installed on the bare Structural Steel frame prior to completion of the lateral-load-resisting system. These elements may increase the potential for lateral loads on the temporary supports. Such temporary supports may also be required to be left in place after the Structural Steel frame has been erected. Special provisions should be made by the Owner's Designated Representative for Construction for these conditions.

7.10.4. All temporary supports that are required for the erection operation and furnished and installed by the Erector shall remain the property of the Erector and shall not be modified, moved or removed without the consent of the Erector. Temporary supports provided by the Erector shall remain in place until the portion of the Structural Steel frame that they brace is complete and the lateral-load-resisting system and connecting diaphragm elements identified by the Owner's Designated Representative for Design in accordance with Section 7.10.1 are installed. Temporary supports that are required to be left in place after the completion of Structural Steel erection shall be removed when no longer needed by the Owner's Designated Representative for Construction and returned to the Erector in good condition.

7.11. Safety Protection

7.11.1. The Erector shall provide floor coverings, handrails, walkways and other safety protection for the Erector's personnel as required by law and the applicable safety regulations. Unless otherwise specified in the Contract Documents, the Erector is permitted to remove such safety protection from areas where the erection operations are completed.

7.11.2. When safety protection provided by the Erector is left in an area for the use of other trades after the Structural Steel erection activity is completed, the Owner's Designated Representative for Construction shall:

(a) Accept responsibility for and maintain this protection;
(b) Indemnify the Fabricator and the Erector from damages that may be incurred from the use of this protection by other trades;
(c) Ensure that this protection is adequate for use by other affected trades;

(d) Ensure that this protection complies with applicable safety regulations when being used by other trades; and,
(e) Remove this protection when it is no longer required and return it to the Erector in the same condition as it was received.

7.11.3. Safety protection for other trades that are not under the direct employment of the Erector shall be the responsibility of the Owner's Designated Representative for Construction.

7.11.4. When permanent steel decking is used for protective flooring and is installed by the Owner's Designated Representative for Construction, all such work shall be scheduled and performed in a timely manner so as not to interfere with or delay the work of the Fabricator or the Erector. The sequence of installation that is used shall meet all safety regulations.

7.11.5. Unless the interaction and safety of activities of others, such as construction by others or the storage of materials that belong to others, are coordinated with the work of the Erector by the Owner's Designated Representative for Construction, such activities shall not be permitted until the erection of the Structural Steel frame or portion thereof is completed by the Erector and accepted by the Owner's Designated Representative for Construction.

7.12. Structural Steel Frame Tolerances

The accumulation of the mill tolerances and fabrication tolerances shall not cause the erection tolerances to be exceeded.

Commentary:
In previous editions of this Code, it was stated that "…variations are deemed to be within the limits of good practice when they do not exceed the cumulative effect of rolling tolerances, fabricating tolerances and erection tolerances." It is recognized in the current provision in this Section that accumulations of mill tolerances and fabrication tolerances generally occur between the locations at which erection tolerances are applied, and not at the same locations.

7.13. Erection Tolerances

Erection tolerances shall be defined relative to member working points and working lines, which shall be defined as follows:

(a) For members other than horizontal members, the member work point shall be the actual center of the member at each end of the shipping piece.
(b) For horizontal members, the working point shall be the actual centerline of the top flange or top surface at each end.
(c) The member working line shall be the straight line that connects the member working points.

The substitution of other working points is permitted for ease of reference, provided they are based upon the above definitions.

The tolerances on Structural Steel erection shall be in accordance with the requirements in Sections 7.13.1 through 7.13.3.

Commentary:

The erection tolerances defined in this Section have been developed through long-standing usage as practical criteria for the erection of Structural Steel. Erection tolerances were first defined in the 1924 edition of this Code in Section 7(f), “Plumbing Up.” With the changes that took place in the types and use of materials in building construction after World War II, and the increasing demand by Architects and Owners for more specific tolerances, AISC adopted new standards for erection tolerances in Section 7(h) of the March 15, 1959 edition of this Code. Experience has proven that those tolerances can be economically obtained.

Differential column shortening may be a consideration in design and construction. In some cases, it may occur due to variability in the accumulation of dead load among different columns (see Figure C–7.1). In other cases, it may be characteristic of the structural system that is employed in the design. Consideration of the effects of differential column shortening may be very important, such as when the slab thickness is reduced, when electrical and other similar fittings mounted on the Structural Steel are intended to be flush with the finished floor and when there is little clearance between bottoms of beams and the tops of door frames or ductwork.

Expansion and contraction in a Structural Steel frame may also be a consideration in the design and construction. Steel will expand or contract approximately 1/8 in. per 100 ft for each change of 15°F [2 mm per 10 000 mm for each change of 15°C] in temperature. This change in length can be assumed to act about the center of rigidity. When anchored to their foundations, end columns will be plumb only when the steel is at normal temperature (see Figure C–7.2). It is therefore necessary to correct field measurements of offsets to the structure from established baselines for the expansion or contraction of the exposed Structural Steel frame. For example, a 200-ft-long [60 000-m-long] building that is plumbed up at 100°F [38°C] should have working points at the tops of the end columns positioned 1/2 in. [14 mm] further apart than the working points at the corresponding bases in order for the columns to be plumb at 70°F [21°C]. Differential temperature effects on column length should also be taken into account in plumbing surveys when tall Structural Steel frames are subjected to sun exposure on one side.

The alignment of lintels, spandrels, wall supports and similar members that are used to connect other building construction units to the Structural Steel frame should have an adjustment of sufficient magnitude to allow for the accumulation of mill tolerances and fabrication tolerances, as well as the erection tolerances. See Figure C–7.3.

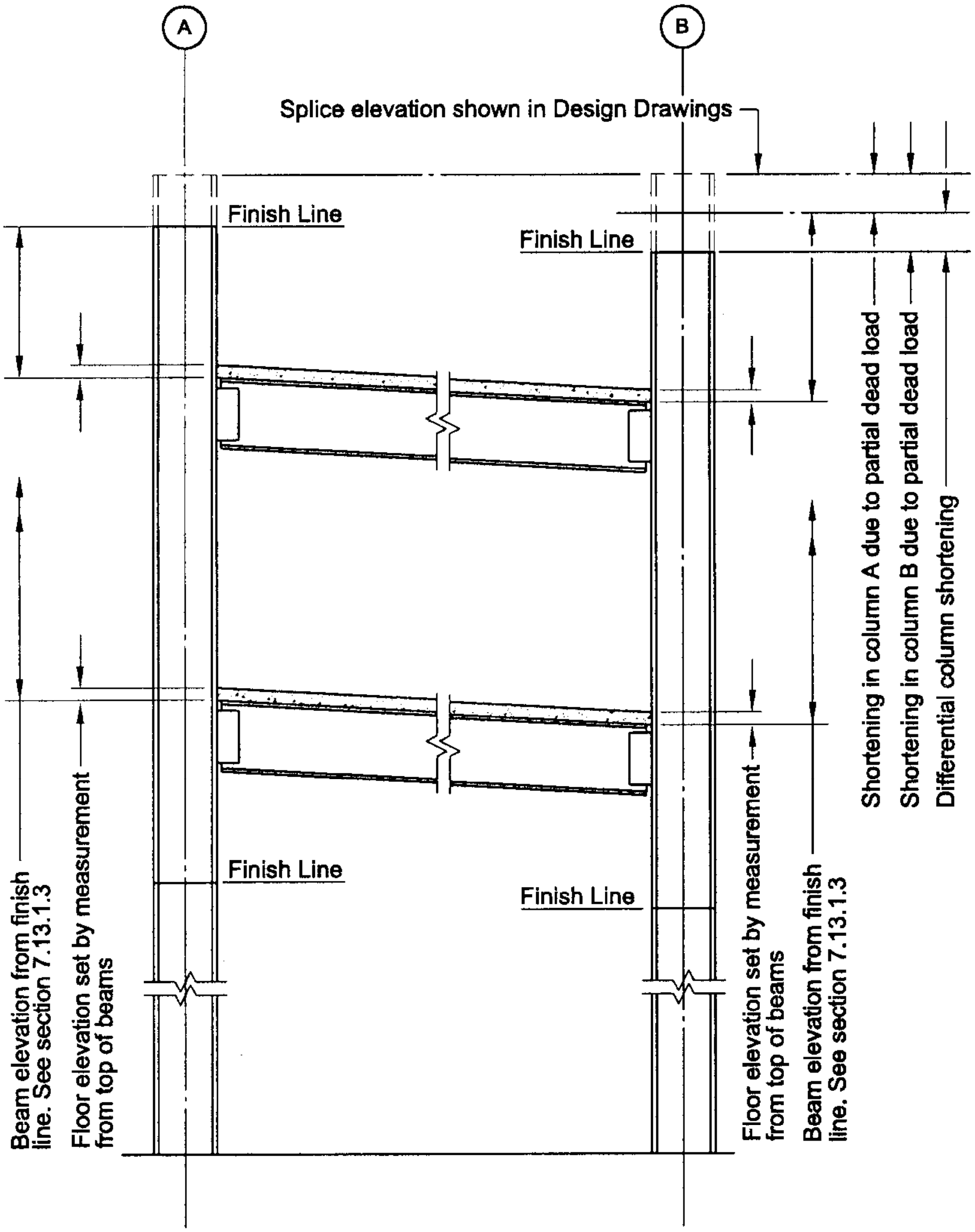
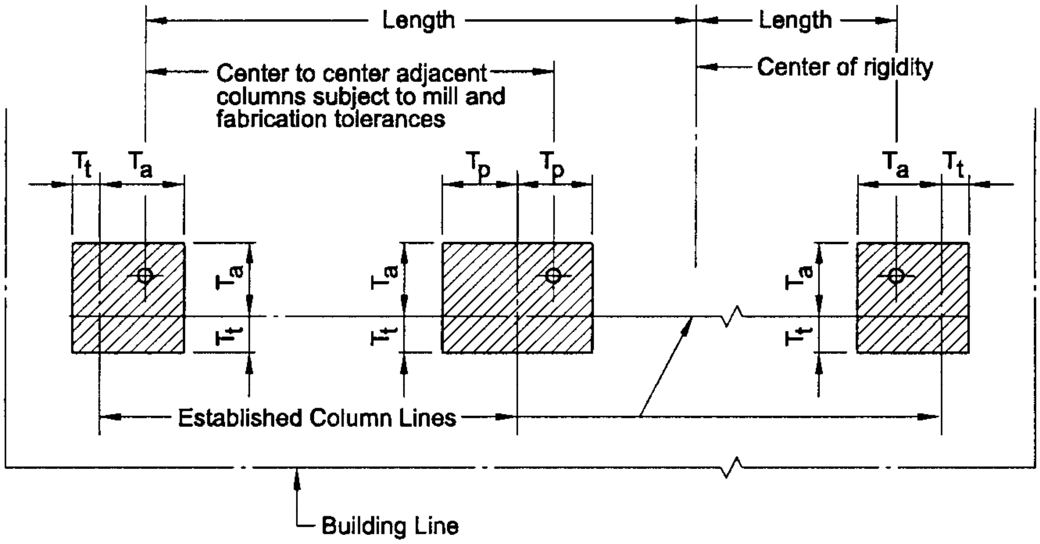
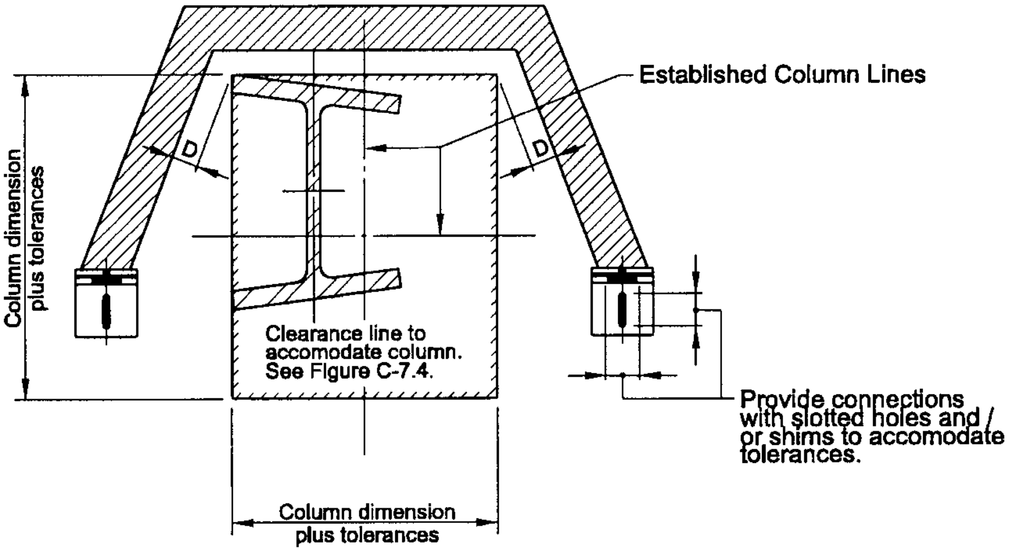


Figure C-7.1. Effects of differential column shortening.

When plumbing columns, apply a temperature adjustment at a rate of 1/8 in. per 100 ft. for each change of 15° F [2 mm per 10 000 mm for each change of 15° C] between the temperature at the time of erection and the working temperature.





If facia joints are set from nearest column finish line, allow ±5/8 in. [16mm] for vertical adjustment. The entity responsible for the facia details must allow for progressive shortening of steel columns.

D= Tolerances required by manufacturer of wall units plus survey tolerances.

Figure C-7.3.Clearance required to accommodate fascia.

Commentary:
The limitations that are described in this Section and illustrated in Figures C–7.4 and C–7.5 make it possible to maintain built-in-place or prefabricated facades in a true vertical plane up to the 20th story, if Connections that provide for 3 in. [75 mm] of adjustment are used. Above the 20th story, the facade may be maintained within 1/16 in. [2 mm] per story with a maximum total deviation of 1 in. [25 mm] from a true vertical plane, if Connections that provide for 3 in. [75 mm] of adjustment are used. Connections that permit adjustments of plus 2 in. [50 mm] to minus 3 in. [75 mm] (5 in. [125 mm] total) will be necessary in cases where it is desired to construct the facade to a true vertical plane above the 20th story.

(c) For an exterior individual column shipping piece, the member working points at any splice level for multi-Tier buildings and at the tops of columns for single-Tier buildings shall fall within a horizontal envelope, parallel to the building line, that is equal to or less than 1 1/2 in. [38 mm] wide for buildings up to 300 ft [90 000 mm] in length. An increase in the width of this horizontal envelope of 1/2 in. [13 mm] is permitted for each additional 100 ft [30 000 m] in length up to a maximum width of 3 in. [75 mm].

Commentary:
This Section limits the position of exterior column working points at any given splice elevation to a narrow horizontal envelope parallel to the building line (see Figure C–7.6). This envelope is limited to a width of 1 1/2 in. [38 mm], normal to the building line, in up to 300 ft [90 000 mm] of building length. The horizontal location of this envelope is not necessarily directly above or below the corresponding envelope at the adjacent splice elevations, but should be within the limitation of the 1 in 500 plumbness tolerance specified for the controlling columns (see Figure C–7.5).

(d) For an exterior column shipping piece, the displacement of member working points from the Established Column Line, parallel to the building line, shall be equal to or less than 2 in. [50 mm] in the first 20 stories. Above this level, an increase in the displacement of 1/16 in. [2 mm] is permitted for each additional story up to a maximum displacement of 3 in. [75 mm] parallel to the building line.

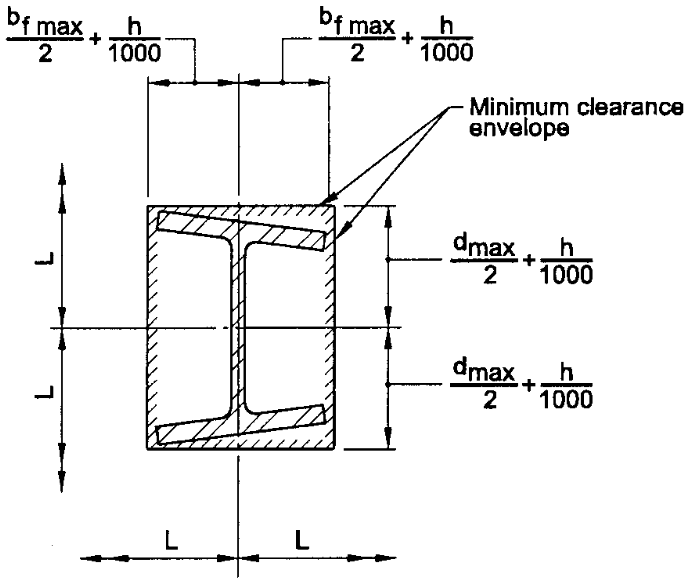
7.13.1.2. For members other than column shipping pieces, the following limitations shall apply:

(a) For a member that consists of an individual, straight shipping piece without field splices, other than a cantilevered member, the variation in alignment shall be acceptable if it is caused solely by variations in column alignment and/or primary supporting member alignment that are within the permissible variations for the fabrication and erection of such members.

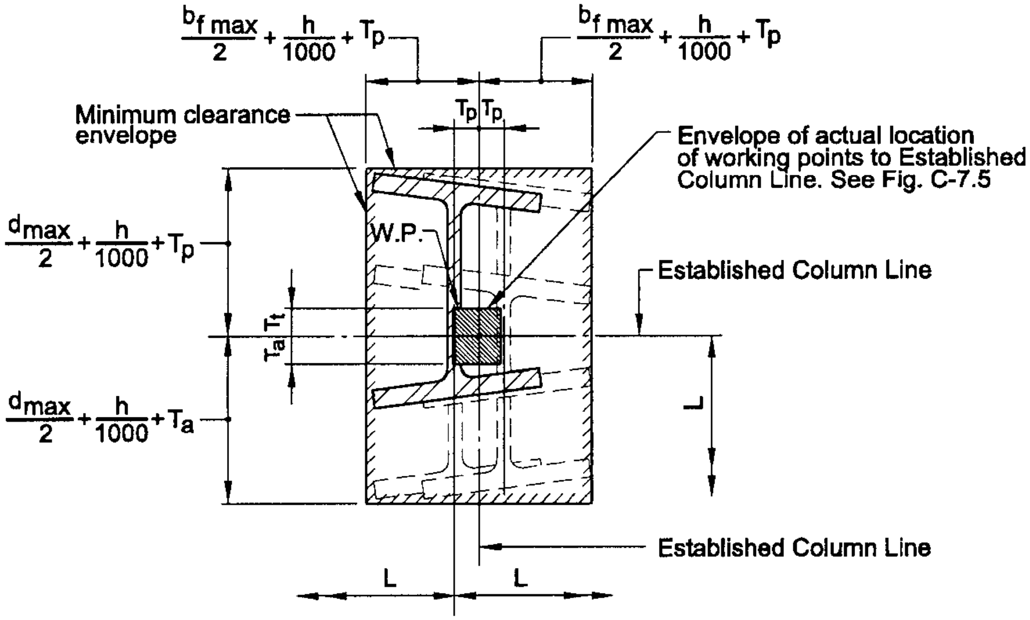
(b) For a member that consists of an individual, straight shipping piece that connects to a column, the variation in the distance from the member working point to the upper finished splice line of the column shall be equal to or less than plus 3/16 in. [5 mm] and minus 5/16 in. [8 mm].

(c) For a member that consists of an individual shipping piece that does not connect to a column, the variation in elevation shall be acceptable if it is caused solely by the variations in the elevations of the supporting members within the permissible variations for the fabrication and erection of those members.

(d) For a member that consists of an individual, straight shipping piece and that is a segment of a field assembled unit containing field splices between points of support, the plumbness, elevation and alignment shall be acceptable if the angular variation of the working line from the plan alignment is equal to or less than 1/500 of the distance between working points.



For enclosures or attachments that may follow column alignment.



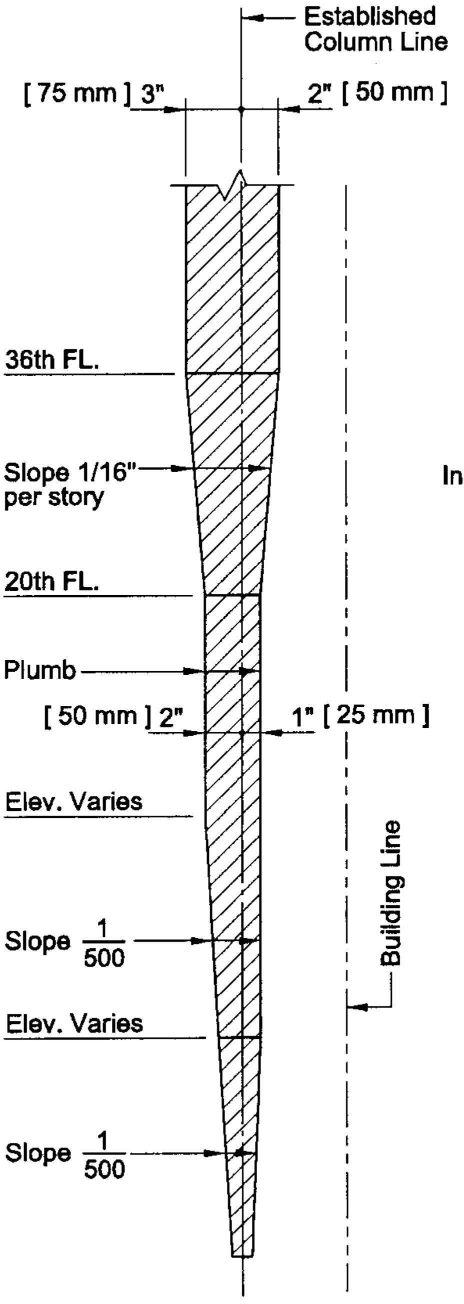
For enclosures or attachments that must be held to precise plan location.

L = Actual center to center of columns = plan dimensions ± column cross section tolerance of columns ± beam length tolerance.

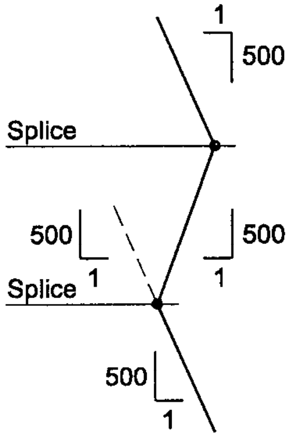
T_a = Plumbness tolerance away from building line (varies, see Fig. C-7.5)

T_t = Plumbness tolerance toward building line (varies, see Fig. C-7.5)

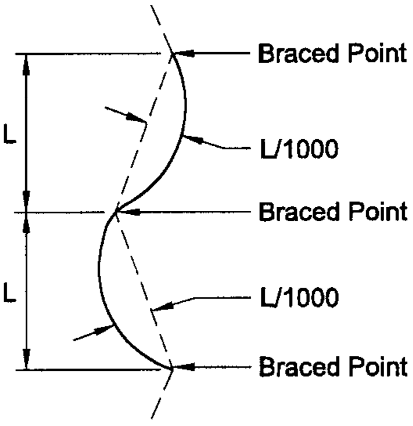
T_p = Plumbness tolerance parallel to building line (=T_a)



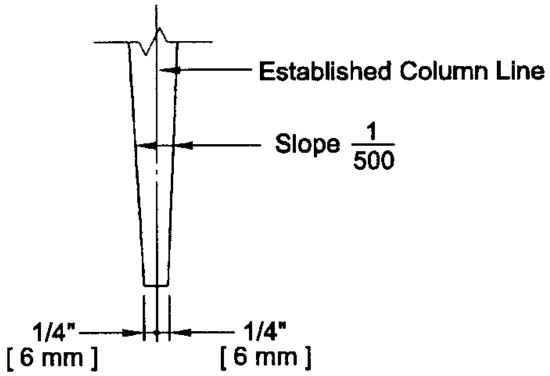
Envelope within which all working points must fall.



Individual pieces within envelope defined at left

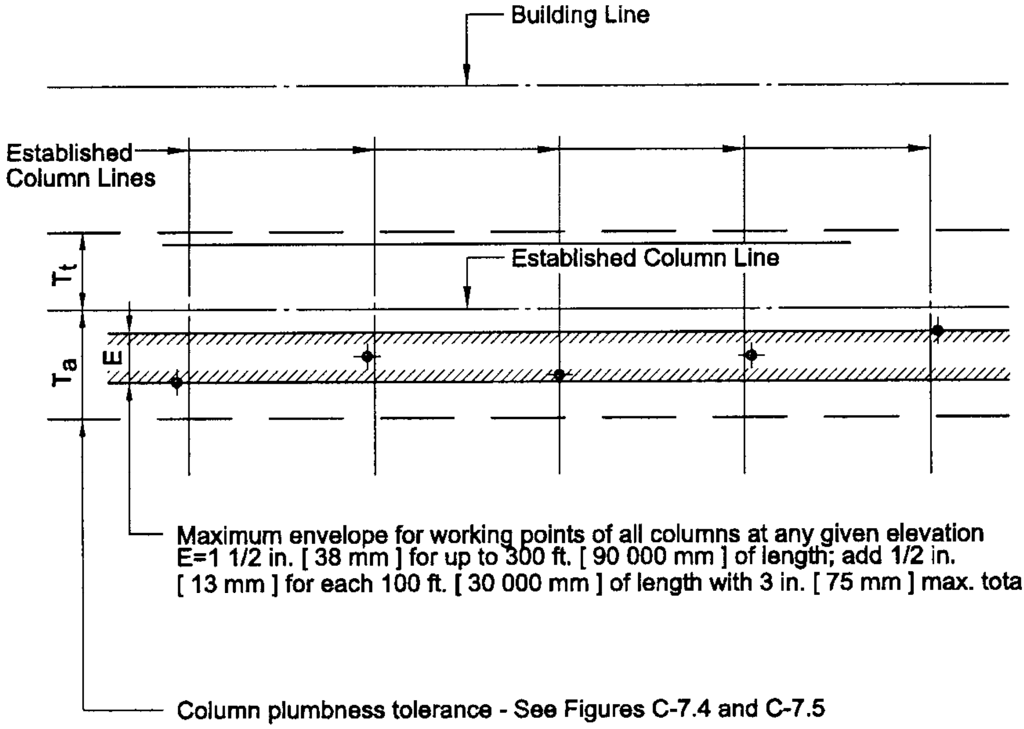


Between brace points (usually stories)



Tolerance on location of W.P. at base.

Note: The plumb line through the base working point for an individual column is not necessarily the precise plan location because Sect. 7.13.1.1 deals only with plumbness tolerances and does not include inaccuracies in location of the Established Column Line, foundations and anchor rods beyond the Erector's control



Maximum envelope for working points of all columns at any given elevation E=1 1/2 in. [38 mm] for up to 300 ft. [90 000 mm] of length; add 1/2 in. [13 mm] for each 100 ft. [30 000 mm] of length with 3 in. [75 mm] max. total.

Column plumbness tolerance - See Figures C-7.4 and C-7.5

-⌖- Indicates column working points

At any splice elevation, envelope "E" is located within the limits T_a and T_t

At any splice elevation, envelope "E" may be located offset from the corresponding envelope at the adjacent splice elevations, above and below, by an amount not greater than $\frac{1}{500}$ of the column length.

Figure C-7.6. Tolerances in plan at any splice elevation of exterior columns.

Commentary:

The angular misalignment of the working line of all fabricated shipping pieces relative to the line between support points of the member as a whole in erected position must not exceed 1 in 500. Note that the tolerance is not stated in terms of a linear displacement at any point and is not to be taken as the overall length between supports divided by 500. Typical examples are shown in Figure C–7.7. Numerous conditions within tolerance for these and other cases are possible. This condition applies to both plan and elevation tolerances.

(g) For a member that is fully assembled in the field in an unstressed condition, the same tolerances shall apply as if fully assembled in the shop.

(h) For a member that is field-assembled, element-by-element in place, temporary support shall be used or an alternative erection plan shall be submitted to the Owner's Designated Representatives for Design and Construction. The tolerance in Section 7.13.1.2(d) shall be met in the supported condition with working points taken at the point(s) of temporary support.

Commentary:
Trusses fabricated and erected as a unit or as an assembly of truss segments normally have excellent controls on vertical position regardless of fabrication and erection techniques. However, a truss fabricated and erected by assembling individual components in place in the field is potentially more sensitive to deflections of the individual truss components and the partially completed work during erection, particularly the chord members. In such a case, the erection process should follow an erection plan that addresses this issue.

7.13.1.3. For members that are identified as Adjustable Items by the Owner's Designated Representative for Design in the Contract Documents, the Fabricator shall provide adjustable Connections for these members to the supporting Structural Steel frame. Otherwise, the Fabricator is permitted to provide non-adjustable Connections. When Adjustable Items are specified, the Owner's Designated Representative for Design shall indicate the total adjustability that is required for the proper alignment of these supports for other trades. The variation in the position and alignment of Adjustable Items shall be as follows:

(a) The variation in the vertical distance from the upper finished splice line of the nearest column to the support location specified in the structural Design Drawings shall be equal to or less than plus or minus 3/8 in. [10 mm].
(b) The variation in the horizontal distance from the established finish line at the particular floor shall be equal to or less than plus or minus 3/8 in. [10 mm].
(c) The variation in vertical and horizontal alignment at the abutting ends of Adjustable Items shall be equal to or less than plus or minus 3/16 in. [5 mm].

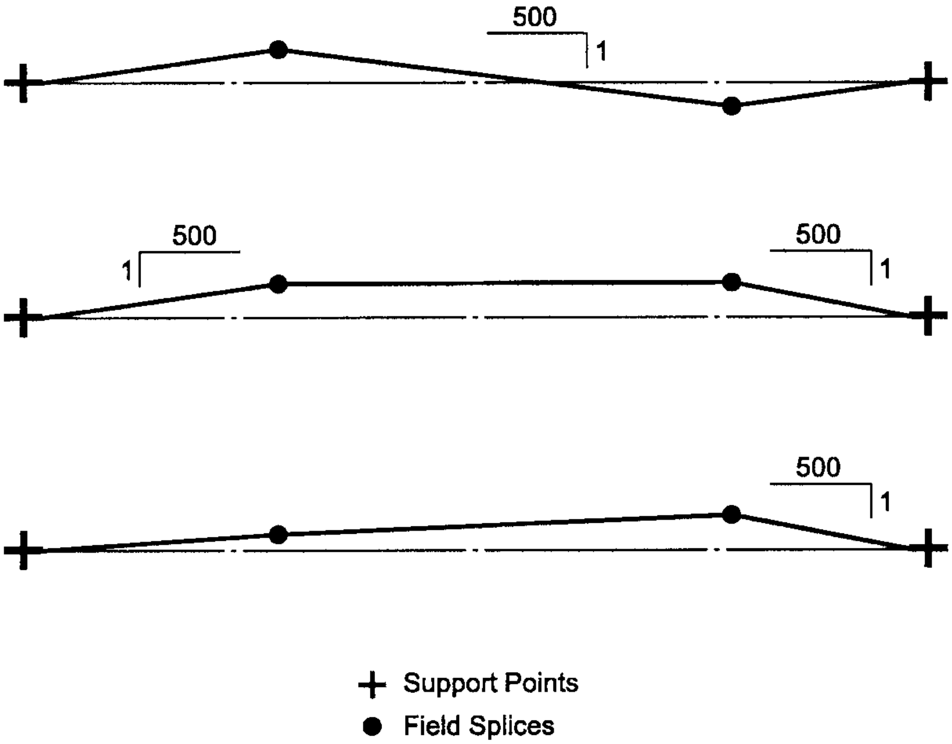


Figure C-7.7. Alignment tolerances for members with field splices.

Commentary:
When the alignment of lintels, wall supports, curb angles, mullions and similar supporting members for the use of other trades is required to be closer than that permitted by the foregoing tolerances for Structural Steel, the Owner's Designated Representative for Design must identify such items in the Contract Documents as Adjustable Items.

7.13.2. In the design of steel structures, the Owner's Designated Representative for Design shall provide for the necessary clearances and adjustments for material furnished by other trades to accommodate the mill tolerances, fabrication tolerances and erection tolerances in this Code for the Structural Steel frame.

Commentary:
In spite of all efforts to minimize inaccuracies, deviations will still exist; therefore, in addition, the designs of prefabricated wall panels, partition panels, fenestrations, floor-to-ceiling door frames and similar elements must provide for clearance and details for adjustment as described in Section 7.13.2. Designs must provide for adjustment in the vertical dimension of prefabricated facade panels that are supported by the Structural Steel frame because the accumulation of shortening of loaded steel columns will result in the unstressed facade supported at each floor level being higher than the Structural Steel framing to

which it must be attached. Observations in the field have shown that where a heavy facade is erected to a greater height on one side of a multistory building than on the other, the Structural Steel framing will be pulled out of alignment. Facades should be erected at a relatively uniform rate around the perimeter of the structure.

7.13.3. Prior to placing or applying any other materials, the Owner's Designated Representative for Construction shall determine that the location of the Structural Steel is acceptable for plumbness, elevation and alignment. The Erector shall be given either timely notice of acceptance by the Owner's Designated Representative for Construction, or a listing of specific items that are to be corrected in order to obtain acceptance. Such notice shall be rendered promptly upon completion of any part of the work and prior to the start of work by other trades that may be supported, attached or applied to the Structural Steel frame.

7.14. Correction of Errors

The correction of minor misfits by moderate amounts of reaming, grinding, welding or cutting, and the drawing of elements into line with drift pins, shall be considered to be normal erection operations. Errors that cannot be corrected using the foregoing means, or that require major changes in member or Connection configuration, shall be promptly reported to the Owner's Designated Representatives for Design and Construction and the Fabricator by the Erector, to enable the responsible entity to either correct the error or approve the most efficient and economical method of correction to be used by others.

Commentary:

As used in this Section, the term “moderate” refers to the amount of reaming, grinding, welding or cutting that must be done on the project as a whole, not the amount that is required at an individual location. It is not intended to address limitations on the amount of material that is removed by reaming at an individual bolt hole, for example, which is limited by the bolt-hole size and tolerance requirements in the AISC and RCSC Specifications.

7.15. Cuts, Alterations and Holes for Other Trades

Neither the Fabricator nor the Erector shall cut, drill or otherwise alter their work, nor the work of other trades, to accommodate other trades, unless such work is clearly specified in the Contract Documents. When such work is so specified, the Owner's Designated Representatives for Design and Construction shall furnish complete information as to materials, size, location and number of alterations in a timely manner so as not to delay the preparation of Shop and Erection Drawings.

7.16. Handling and Storage

The Erector shall take reasonable care in the proper handling and storage of the Structural Steel during erection operations to avoid the accumulation of excess dirt and foreign matter. The Erector shall not be responsible for the removal from the Structural Steel of dust, dirt or other foreign matter that may accumulate during erection as the result of job-site conditions or exposure to the elements. The Erector shall handle and store all bolts, nuts, washers and related fastening products in accordance with the requirements of the RCSC Specification.

Commentary:
During storage, loading, transport, unloading and erection, blemish marks caused by slings, chains, blocking, tie-downs, etc., occur in varying degrees. Abrasions caused by handling or cartage after painting are to be expected. It must be recognized that any shop-applied coating, no matter how carefully protected, will require touching-up in the field. Touching-up of these blemished areas is the responsibility of the contractor performing the field touch-up or field painting.

The Erector is responsible for the proper storage and handling of fabricated Structural Steel at the job site during erection. Shop-painted Structural Steel that is stored in the field pending erection should be kept free of the ground and positioned so as to minimize the potential for water retention. The Owner or Owner's Designated Representative for Construction is responsible for providing suitable job-site conditions and proper access so that the Fabricator/Erector may perform its work.

Job-site conditions are frequently muddy, sandy, dusty or a combination thereof during the erection period. Under such conditions it may be impossible to store and handle the Structural Steel in such a way as to completely avoid any accumulation of mud, dirt or sand on the surface of the Structural Steel, even though the Fabricator and the Erector manages to proceed with their work.

Repairs of damage to painted surfaces and/or removal of foreign materials due to adverse job-site conditions are outside the scope of responsibility of the Fabricator and the Erector when reasonable attempts at proper handling and storage have been made.

7.17. Field Painting

Neither the Fabricator nor the Erector is responsible to paint field bolt heads and nuts or field welds, nor to touch up abrasions of the shop coat, nor to perform any other field painting.

7.18. Final Cleaning Up

Upon the completion of erection and before final acceptance, the Erector shall remove all of the Erector's falsework, rubbish and temporary buildings.

SECTION 8. QUALITY ASSURANCE

8.1. General

8.1.1. The Fabricator shall maintain a quality assurance program to ensure that the work is performed in accordance with the requirements in this Code, the AISC Specification and the Contract Documents. The Fabricator shall have the option to use the AISC Quality Certification Program to establish and administer the quality assurance program.

Commentary:
The AISC Quality Certification Program confirms to the construction industry that a certified Structural Steel fabrication shop has the capability by reason of commitment, personnel, organization, experience, procedures, knowledge and equipment to produce fabricated Structural Steel of the required quality for a given category of work. The AISC Quality Certification Program is not intended to involve inspection and/or judgment of product quality on individual projects. Neither is it intended to guarantee the quality of specific fabricated Structural Steel products.

8.1.2. The Erector shall maintain a quality assurance program to ensure that the work is performed in accordance with the requirements in this Code, the AISC Specification and the Contract Documents. The Erector shall be capable of performing the erection of the Structural Steel, and shall provide the equipment, personnel and management for the scope, magnitude and required quality of each project. The Erector shall have the option to use the AISC Erector Certification Program to establish and administer the quality assurance program.

Commentary:
The AISC Erector Certification Program confirms to the construction industry that a certified Structural Steel Erector has the capability by reason of commitment, personnel, organization, experience, procedures, knowledge and equipment to erect fabricated Structural Steel to the required quality for a given category of work. The AISC Erector Certification Program is not intended to involve inspection and/or judgment of product quality on individual projects. Neither is it intended to guarantee the quality of specific erected Structural Steel products.

8.1.3. When the Owner requires more extensive quality assurance or independent inspection by qualified personnel, or requires that the Fabricator be certified under the AISC Quality Certification Program and/or requires that the Erector be certified under the AISC Erector Certification Program, this shall be clearly stated in the Contract Documents, including a definition of the scope of such inspection.

8.2. Inspection of Mill Material

Certified mill test reports shall constitute sufficient evidence that the mill product satisfies material order requirements. The Fabricator shall make a visual inspection of material that is received from the mill, but need not perform any material tests unless the Owner's Designated Representative for Design specifies in the Contract Documents that additional testing is to be performed at the Owner’s expense.

8.3. Non-Destructive Testing

When non-destructive testing is required, the process, extent, technique and standards of acceptance shall be clearly specified in the Contract Documents.

8.4. Surface Preparation and Shop Painting Inspection

Inspection of surface preparation and shop painting shall be planned for the acceptance of each operation as the Fabricator completes it. Inspection of the paint system, including material and thickness, shall be made promptly upon completion of the paint application. When wet-film thickness is to be inspected, it shall be measured during the application.

8.5. Independent Inspection

When inspection by personnel other than those of the Fabricator and/or Erector is specified in the Contract Documents, the requirements in Sections 8.5.1 through 8.5.6 shall be met.

8.5.1. The Fabricator and the Erector shall provide the Inspector with access to all places where the work is being performed. A minimum of 24 hours notification shall be given prior to the commencement of work.

8.5.2. Inspection of shop work by the Inspector shall be performed in the Fabricator’s shop to the fullest extent possible. Such inspections shall be timely, in-sequence and performed in such a manner as will not disrupt fabrication operations and will permit the repair of non-conforming work prior to any required painting while the material is still in-process in the fabrication shop.

8.5.3. Inspection of field work shall be promptly completed without delaying the progress or correction of the work.

8.5.4. Rejection of material or workmanship that is not in conformance with the Contract Documents shall be permitted at any time during the progress of the work. However, this provision shall not relieve the Owner or the Inspector of the obligation for timely, in-sequence inspections.

8.5.5. The Fabricator, Erector, and Owner's Designated Representatives for Design and Construction shall be informed of deficiencies that are noted by the Inspector promptly after the inspection. Copies of all reports prepared by the Inspector shall be promptly given to the Fabricator, Erector and Owner's Designated Representatives for Design and Construction. The necessary corrective work shall be performed in a timely manner.

8.5.6. The Inspector shall not suggest, direct, or approve the Fabricator or Erector to deviate from the Contract Documents or the approved Shop and Erection Drawings, or approve such deviation, without the written approval of the Owner's Designated Representatives for Design and Construction.

SECTION 9. CONTRACTS

9.1. Types of Contracts

9.1.1. For contracts that stipulate a lump sum price, the work that is required to be performed by the Fabricator and the Erector shall be completely defined in the Contract Documents.

9.1.2. For contracts that stipulate a price per pound, the scope of work that is required to be performed by the Fabricator and the Erector, the type of materials, the character of fabrication and the conditions of erection shall be based upon the Contract Documents, which shall be representative of the work to be performed.

9.1.3. For contracts that stipulate a price per item, the work that is required to be performed by the Fabricator and the Erector shall be based upon the quantity and the character of the items that are described in the Contract Documents.

9.1.4. For contracts that stipulate unit prices for various categories of Structural Steel, the scope of work that is required to be performed by the Fabricator and the Erector shall be based upon the quantity, character and complexity of the items in each category as described in the Contract Documents, and shall also be representative of the work to be performed in each category.

9.2. Calculation of Weights

Unless otherwise specified in the contract, for contracts stipulating a price per pound for fabricated Structural Steel that is delivered and/or erected, the quantities of materials for payment shall be determined by the calculation of the gross weight of materials as shown in the Shop Drawings.

Commentary:
The standard procedure for calculation of weights that is described in this Code meets the need for a universally acceptable system for defining “pay weights” in contracts based upon the weight of delivered and/or erected materials. These procedures permits the Owner to easily and accurately evaluate price-per-pound proposals from potential suppliers and enables all parties to a contract to have a clear and common understanding of the basis for payment.

The procedure in this Code affords a simple, readily understood method of calculation that will produce pay weights that are consistent throughout the industry and that may be easily verified by the Owner. While this procedure does not produce actual weights, it can be used by purchasers and suppliers to define a widely accepted basis for bidding and contracting for Structural Steel. However, any other system can be used as the basis for a contractual agreement. When other systems are used, both the supplier and the purchaser should clearly understand how the alternative procedure is handled.

9.2.1. The unit weight of steel shall be taken as 490 lb/ft^3 [7 850 kg/m^3]. The unit weight of other materials shall be in accordance with the manufacturer's published data for the specific product.

9.2.2. The weights of Standard Structural Shapes, plates and bars shall be calculated on the basis of Shop Drawings that show the actual quantities and dimensions of material to be fabricated, as follows:

(a) The weights of all Standard Structural Shapes shall be calculated using the nominal weight per ft [mass per m] and the detailed overall length.
(b) The weights of plates and bars shall be calculated using the detailed overall rectangular dimensions.
(c) When parts can be economically cut in multiples from material of larger dimensions, the weight shall be calculated on the basis of the theoretical rectangular dimensions of the material from which the parts are cut.
(d) When parts are cut from Standard Structural Shapes, leaving a non-standard section that is not useable on the same contract, the weight shall be calculated using the nominal weight per ft [mass per m] and the overall length of the Standard Structural Shapes from which the parts are cut.
(e) Deductions shall not be made for material that is removed for cuts, copes, clips, blocks, drilling, punching, boring, slot milling, planing or weld joint preparation.

9.2.3. The items for which weights are shown in tables in the AISC Manual of Steel Construction shall be calculated on the basis of the tabulated weights shown therein.

9.2.4. The weights of items that are not shown in tables in the AISC Manual of Steel Construction shall be taken from the manufacturer's catalog and the manufacturer's shipping weight shall be used.

Commentary:
Many items that are weighed for payment purposes are not tabulated with weights in the AISC Manual of Steel Construction. These include, but are not limited to, Anchor Rods, clevises, turnbuckles, sleeve nuts, recessed-pin nuts, cotter pins and similar devices.

9.2.5. The weights of shop or field weld metal and protective coatings shall not be included in the calculated weight for the purposes of payment.

9.3. Revisions to the Contract Documents

Revisions to the Contract Documents shall be confirmed by change order or extra work order. Unless otherwise noted, the issuance of a revision to the

Contract Documents shall constitute authorization by the Owner that the revision is Released for Construction. The contract price and schedule shall be adjusted in accordance with Sections 9.4 and 9.5.

9.4. Contract Price Adjustment

9.4.1. When the scope of work and responsibilities of the Fabricator and the Erector are changed from those previously established in the Contract Documents, an appropriate modification of the contract price shall be made. In computing the contract price adjustment, the Fabricator and the Erector shall consider the quantity of work that is added or deleted, the modifications in the character of the work and the timeliness of the change with respect to the status of material ordering, detailing, fabrication and erection operations.

Commentary:
The fabrication and erection of Structural Steel is a dynamic process. Typically, material is being acquired at the same time that the Shop and Erection Drawings are being prepared. Additionally, the fabrication shop will normally fabricate pieces in the order that the Structural Steel is being shipped and erected.

Items that are revised or placed on hold generally upset these relationships and can be very disruptive to the detailing, fabricating and erecting processes. The provisions in Sections 3.5, 4.4.2 and 9.3 are intended to minimize these disruptions so as to allow work to continue. Accordingly, it is required in this Code that the reviewer of requests for contract price adjustments recognize this and allow compensation to the Fabricator and the Erector for these inefficiencies and for the materials that are purchased and the detailing, fabrication and erection that has been performed, when affected by the change.

9.4.2. Requests for contract price adjustments shall be presented by the Fabricator and/or the Erector in a timely manner and shall be accompanied by a description of the change that is sufficient to permit evaluation and timely approval by the Owner.

9.4.3. Price-per-pound and price-per-item contracts shall provide for additions or deletions to the quantity of work that are made prior to the time the work is Released for Construction. When changes are made to the character of the work at any time, or when additions and/or deletions are made to the quantity of the work after it is released for detailing, fabrication or erection, the contract price shall be equitably adjusted.

9.5. Scheduling

9.5.1. The contract schedule shall state when the Design Drawings will be Released for Construction, if the Design Drawings are not available at the time of bidding, and when the job site, foundations, piers and abutments will be ready, free from obstructions and accessible to the Erector, so that erection can start at the designated time and continue without interference or delay caused by the Owner's Designated Representative for Construction or other trades.

9.5.2. The Fabricator and the Erector shall advise the Owner's Designated Representatives for Design and Construction, in a timely manner, of the effect any revision has on the contract schedule.

9.5.3. If the fabrication or erection is significantly delayed due to revisions to the requirements of the contract, or for other reasons that are the responsibility of others, the Fabricator and/or Erector shall be compensated for the additional costs incurred.

9.6. Terms of Payment

The Fabricator shall be paid for Mill Materials and fabricated product that is stored off the job site. Other terms of payment for the contract shall be outlined in the Contract Documents.

Commentary:

These terms include such items as progress payments for material, fabrication, erection, retainage, performance and payment bonds and final payment. If a performance or payment bond, paid for by the Owner, is required by contract, no retainage shall be required.

SECTION 10. ARCHITECTURALLY EXPOSED STRUCTURAL STEEL

10.1. General Requirements

When members are specifically designated as "Architecturally Exposed Structural Steel" or "AESS" in the Contract Documents, the requirements in Sections 1 through 9 shall apply as modified in Section 10. AESS members or components shall be fabricated and erected with the care and dimensional tolerances that are stipulated in Sections 10.2 through 10.4. The following additional information shall be provided in the Contract Documents when AESS is specified:

(a) Specific identification of members or components that are AESS;
(b) Fabrication and/or erection tolerances that are to be more restrictive than provided for in this Section, if any; and,
(c) Requirements, if any, of a mock-up panel or components for inspection and acceptance standards prior to the start of fabrication.

Commentary:
This Section of this Code defines additional requirements that apply only to members that are specifically designated by the Contract Documents as "Architecturally Exposed Structural Steel" (AESS). The rapidly increasing use of exposed Structural Steel as a medium of architectural expression has given rise to a demand for closer dimensional tolerances and smoother finished surfaces than required for ordinary Structural Steel framing.

This Section of this Code establishes standards for these requirements that take into account both the desired finished appearance and the abilities of the fabrication shop to produce the desired product. It should be pointed out that the term "Architecturally Exposed Structural Steel" (AESS), as covered in this Section, must be specified in the Contract Documents if the Fabricator is required to meet the fabricating standards in this Section, and applies only to that portion of the Structural Steel so identified.

AESS requirements usually involve significant cost in excess of that for Structural Steel that is fabricated in the absence of an AESS requirement. Therefore, the designation AESS should be applied rationally, with visual acceptance criteria that are appropriate for the distance at which the exposed element will be viewed in the completed structure. In order to avoid misunderstandings and to hold costs to a minimum, only those Structural Steel surfaces and Connections that will remain exposed and subject to normal view by pedestrians or occupants of the completed structure should be designated as AESS.

10.2. Fabrication

10.2.1. The permissible tolerances for out-of-square or out-of-parallel, depth, width and symmetry of rolled shapes shall be as specified in ASTM A6/A6M. Unless otherwise specified in the Contract Documents, the exact matching of abutting cross-sectional configurations shall not be necessary. The as-fabricated straightness tolerances of members shall be one-half of the standard camber and sweep tolerances in ASTM A6/A6M.

10.2.2. The tolerances on overall profile dimensions of members that are built-up from a series of Standard Structural Shapes, plates and/or bars by welding shall be taken as the accumulation of the variations that are permitted for the component parts in ASTM A6/A6M. The as-fabricated straightness tolerances for the member as a whole shall be one-half the standard camber and sweep tolerances for rolled shapes in ASTM A6/A6M.

10.2.3. Unless specific visual acceptance criteria for Weld Show-Through are specified in the Contract Documents, the members or components shall be acceptable as produced.

Commentary:
Weld Show-Through is generally a function of weld size and material thickness.

10.2.4. All copes, miters and cuts in surfaces that are exposed to view shall be made with uniform gaps of 1/8 in. [3 mm] if shown as open joints, or in reasonable contact if shown without gap.

10.2.5. All welds that are exposed to view shall be visually acceptable if they meet the requirements in AWS D1.1, except all groove and plug welds that are exposed to view shall not project more than 1/16 in. [2 mm] above the exposed surface. Finishing or grinding of welds shall not be necessary, unless such treatment is required to provide for clearances or fit of other components.

10.2.6. Erection marks or other painted marks shall not be made on those surfaces of weathering steel AESS members that are to be exposed in the completed structure. Unless otherwise specified in the Contract Documents, the Fabricator shall clean weathering steel AESS members to meet the requirements of SSPC-SP6.

10.2.7. Stamped or raised manufacturer's identification marks shall not be filled, ground or otherwise removed.

10.2.8. Seams of hollow structural sections shall be acceptable as produced. Seams shall be oriented away from view or as directed in the Contract Documents.

10.3. Delivery of Materials

The Fabricator shall use special care to avoid bending, twisting or otherwise distorting the Structural Steel.

10.4. Erection

10.4.1. The Erector shall use special care in unloading, handling and erecting the Structural Steel to avoid marking or distorting the Structural Steel. Care shall also be taken to minimize damage to any shop paint. If temporary braces or erection clips are used, care shall be taken to avoid the creation of unsightly surfaces upon removal. Tack welds shall be ground smooth and holes shall be filled with weld metal or body solder and smoothed by grinding or filing. The Erector shall plan and execute all operations in such a manner that the close fit and neat appearance of the structure will not be impaired.

10.4.2. Unless otherwise specified in the Contract Documents, AESS members and components shall be plumbed, leveled and aligned to a tolerance that is one-half that permitted for non-AESS members. To accommodate these erection tolerances for AESS, the Owner's Designated Representative for Design shall specify Connections between AESS members and non-AESS members, masonry, concrete and other supports as Adjustable Items, in order to provide the Erector with means for adjustment.

10.4.3. When AESS is backed with concrete, the Owner's Designated Representative for Construction shall provide sufficient shores, ties and strongbacks to prevent sagging, bulging or similar deformation of the AESS members due to the weight and pressure of the wet concrete.

APPENDIX A. DIGITAL BUILDING PRODUCT MODELS

The provisions in this Appendix shall apply when the contract documents indicate that a three-dimensional digital building product model replaces contract drawings and is to be used as the primary means of designing, representing, and exchanging structural steel data for the project. When this is the case, all references to the Design Drawings in this Code shall instead apply to the Design Model, and all references to the Shop and Erection Drawings in the Code shall instead apply to the Manufacturing Model. The CIS/2 Logical Product Model shall be used as the building product model for structural steel.

If the primary means of project communication reverts from a model-based system to a paper-based system, the requirements in this Code other than in this Appendix shall apply.

Commentary:
Current technology permits the transfer of three-dimensional digital building product model data among the design and construction teams for a project. Over the last several years, designers and fabricators have used CIS/2 as a standard format in the exchange of building product models representing the steel structure. This Appendix facilitates the use of this technology in the design and construction of steel structures, and eliminates any interpretation of this Code that might be construed to prohibit or inhibit the use of this technology. While the technology is new and there is no long-established standard of practice, it is the intent in this Appendix to provide guidance for its use.

APPENDIX A. GLOSSARY

Add the following definitions to the Glossary:

Building Product Model. A digital information structure of the objects making up a building, capturing the form, function, behavior and relations of the parts and assemblies within one or more building systems. A building product model can be implemented in multiple ways, including as an ASCII file or as a database. The data in the model is created, manipulated, evaluated, reviewed and presented using computer-based design, engineering, and manufacturing applications. Traditional two-dimensional drawings may be one of many reports generated by the building product model (see Eastman, Charles M.: Building Product Models: Computer Environments Supporting Design and Construction; 1999 by CRC Press).

CIS/2 (CIMSteel Integration Standards/Version 2). The specification providing the building product model for structural steel and format for electronic data interchange (EDI) among software applications dealing with steel design, analysis, and manufacturing.

Logical Product Model (LPM). The CIS/2 building product model, which supports the engineering of low-, medium- and high-rise construction, in domestic, commercial and industrial contexts. All elements of the structure are covered, including main and secondary framing and connections. The components used can be of any variety of structural shape or element.

The LPM addresses the exchange of data between structural steel applications. It is meant to support a heterogeneous set of applications over a fairly broad portion of the steel lifecycle. It is organized around three different sub-models: the Analysis Model (data represented in structural analysis), the Design Model (data represented in frame design layout) and the Manufacturing Model (data represented in detailing for fabrication).

Data Management Conformance (DMC). The capability of the CIMSteel model to include optional data entities for managing and tracking additions, deletions and modifications to a model, including who made the change and when the change was made for all data changes.

A1.2. Referenced Specifications, Codes and Standards

Add the following reference to Section 1.2:

CIMSteel Integration Standards Release 2: Second Edition P265: CIS/2.1: Volumes 1 through 4.

A3. DESIGN DRAWINGS AND SPECIFICATIONS

In addition to the requirements in Section 3, the following requirements shall apply to the Design Model:

A3.1. Design Model

The Design Model shall:

(a) Consist of Data Management Conformance Classes.
(b) Contain Analysis Model data so as to include load calculations as specified in the Contract Documents.
(c) Include entities that fully define each steel element and the extent of detailing of each element, as would be recorded on equivalent set of structural steel design drawings.
(d) Include all steel elements identified in the Contract Documents as well as any other entities required for strength and stability of the completely erected structure.
(e) Govern over all other forms of information, including drawings, sketches, etc.

A3.2. LPM Administration

The Owner shall designate an Administrator for the LPM, who shall:

(a) Control the LPM by providing appropriate access privileges (read, write, etc) to all relevant parties.
(b) Maintain the security of the LPM.
(c) Guard against data loss of the LPM.
(d) Be responsible for updates and revisions to the LPM as they occur.
(e) Inform all appropriate parties as to changes to the LPM.

Commentary:
When a project is designed and constructed using EDI, it is imperative that an individual entity on the team be responsible for maintaining the LPM. This is to assure protection of data through proper backup, storage and security and to provide coordination of the flow of information to all team members when information is added to the model. Team members exchange information to revise the model with this Administrator. The Administrator will validate all changes to the LPM. This is to assure proper tracking and control of revisions.

This Administrator can be one of the design team members such as an Architect, Structural Engineer or a separate entity on the design team serving this purpose. The Administrator can also be the Fabricator's detailer or a separate entity on the construction team serving this purpose.

A4.3. Fabricator Responsibility

In addition to the requirements in Section 4.3, the following requirements shall apply:

When the Design Model is used to develop the Manufacturing Model the fabricator shall accept the information under the following conditions:

(a) When the design information is to be conveyed to the Fabricator by way of the Design Model, in the event of a conflict between the model and the Design Drawings, the Design Model will control.
(b) The ownership of the information added to the LPM in the Manufacturing Model should be defined in the Contract Documents. In the absence of terms for ownership regarding the information added by the Fabricator to the LPM in the Contract Documents, the ownership will belong to the Fabricator.
(c) During the development of the Manufacturing Model, as member locations are adjusted to convert the modeled parts from a Design Model, these relocations will only be done with the approval of the Owner's Designated Representative for Design.
(d) The Fabricator and Erector shall accept the use of the LPM and Design Model under the same conditions as set forth in Paragraph 4.3 with regard to CAD files, except as modified in A4.3 above.

A4.4. Approval

In addition to the requirements in Section 4.4, the following requirements shall apply:

When the approval of the detailed material is to be done by the use of the Manufacturing Model the version of the submitted model shall be identified. The approver shall annotate the Manufacturing Model with approval comments attached to the individual elements as specified in the CIS/2 standard. As directed by the approval comment the Fabricator will reissue the Manufacturing Model for re-review and the version of the model submitted will be tracked as previously defined.

Commentary:
Approval of the Manufacturing Model by the Owner's Designated Representative for Design can replace the approval of actual shop and erection drawings. For this method to be effective, a system must be in place to record review, approval, correction and final release of the Manufacturing Model for fabrication of structural steel. The versions of the model must be tracked, and review comments and approvals permanently attached to the versions of the model to the same extent as such data is maintained with conventional hard copy approvals. The CIS/2 standard provides this level of tracking.

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PART 17

MISCELLANEOUS DATA AND MATHEMATICAL INFORMATION

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Table 17–1

SI Equivalents of Standard U.S. Shape Profiles

W Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
W44×335	W1100×499	W36×256	W920×381	W27×539	W690×802
×290	×433	×232	×345	×368	×548
×262	×390	×210	×313	×336	×500
×230	×343	×194	×289	×307	×457
W40×593	W1000×883	×182	×271	×281	×419
×503	×748	×170	×253	×258	×384
×431	×642	×160	×238	×235	×350
×397	×591	×150	×223	×217	×323
×372	×554	×135	×201	×194	×289
×362	×539	W33×387	W840×576	×178	×265
×324	×483	×354	×527	×161	×240
×297	×443	×318	×473	×146	×217
×277	×412	×291	×433	W27×129	W690×192
×249	×371	×263	×392	×114	×170
×215	×321	×241	×359	×102	×152
×199	×296	×221	×329	×94	×140
W40×392	W1000×584	×201	×299	×84	×125
×331	×494	W33×169	W840×251	W24×370	W610×551
×327	×486	×152	×226	×335	×498
×294	×438	×141	×210	×306	×455
×278	×415	×130	×193	×279	×415
×264	×393	×118	×176	×250	×372
×235	×350	W30×391	W760×582	×229	×341
×211	×314	×357	×531	×207	×307
×183	×272	×326	×484	×192	×285
×167	×249	×292	×434	×176	×262
×149	×222	×261	×389	×162	×241
W36×800	W920×1191	×235	×350	×146	×217
×652	×970	×211	×314	×131	×195
×529	×787	×191	×284	×117	×174
×487	×725	×173	×257	×104	×155
×441	×656	W30×148	W760×220	W24×103	W610×153
×395	×588	×132	×196	×94	×140
×361	×537	×124	×185	×84	×125
×330	×491	×116	×173	×76	×113
×302	×449	×108	×161	×68	×101
×282	×420	×99	×147	W24×62	W610×92
×262	×390	×90	×134	×55	×82
×247	×368				
×231	×345				

Table 17–1 (continued)

SI Equivalents of Standard U.S. Shape Profiles

W Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W21×201	W530×300
×182	×272
×166	×248
×147	×219
×132	×196
×122	×182
×111	×165
×101	×150
W21×93	W530×138
×83	×123
×73	×109
×68	×101
×62	×92
×55	×82
×48	×72
W21×57	W530×85
×50	×74
×44	×66
W18×311	W460×464
×283	×421
×258	×384
×234	×349
×211	×315
×192	×286
×175	×260
×158	×235
×143	×213
×130	×193
×119	×177
×106	×158
×97	×144
×86	×128
×76	×113
W18×71	W460×106
×65	×97
×60	×89
×55	×82
×50	×74
W18×46	W460×68
×40	×60
×35	×52

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W16×100	W410×149
×89	×132
×77	×114
×67	×100
W16×57	W410×85
×50	×75
×45	×67
×40	×60
×36	×53
W16×31	W410×46.1
×26	×38.8
W14×730	W360×1086
×665	×990
×605	×900
×550	×818
×500	×744
×455	×677
×426	×634
×398	×592
×370	×551
×342	×509
×311	×463
×283	×421
×257	×382
×233	×347
×211	×314
×193	×287
×176	×262
×159	×237
×145	×216
W14×132	W360×196
×120	×179
×109	×162
×99	×147
×90	×134
W14×82	W360×122
×74	×110
×68	×101
×61	×91

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
W14×53	W360×79
×48	×72
×43	×64
W14×38	W360×58
×34	×51
×30	×44.6
W14×26	W360×39
×22	×32.9
W12×336	W310×500
×305	×454
×279	×415
×252	×375
×230	×342
×210	×313
×190	×283
×170	×253
×152	×226
×136	×202
×120	×179
×106	×158
×96	×143
×87	×129
×79	×117
×72	×107
×65	×97
W12×58	W310×86
×53	×79
W12×50	W310×74
×45	×67
×40	×60
W12×35	W310×52
×30	×44.5
×26	×38.7
W12×22	W310×32.7
×19	×28.3
×16	×23.8
×14	×21.0

Table 17–1 (continued)

SI Equivalents of Standard U.S. Shape Profiles

W Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
W10×112	W250×167	W10×19	W250×28.4	W8×15	W200×22.5
×100	×149	×17	×25.3	×13	×19.3
×88	×131	×15	×22.3	×10	×15.0
×77	×115	×12	×17.9		
×68	×101			W6×25	W150×37.1
×60	×89	W8×67	W200×100	×20	×29.8
×54	×80	×58	×86	×15	×22.5
×49	×73	×48	×71		
		×40	×59	W6×16	W150×24.0
W10×45	W250×67	×35	×52	×12	×18.0
×39	×58	×31	×46.1	×9	×13.5
×33	×49.1			×8.5	×13.0
		W8×28	W200×41.7		
W10×30	W250×44.8	×24	×35.9	W5×19	W130×28.1
×26	×38.5			×16	×23.8
×22	×32.7	W8×21	W200×31.3		
		×18	×26.6	W4×13	W100×19.3

Table 17–2
SI Equivalents of Standard U.S. Shape Profiles
M, S and HP Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
M12.5×12.4	M318×18.5
×11.6	×17.3
M12×11.8	M310×17.6
×10.8	×16.1
M12×10	M310×14.9
M10×9	M250×13.4
×8	×11.9
M10×7.5	M250×11.2
M8×6.5	M200×9.7
×6.2	×9.2
M6×4.4	M150×6.6
×3.7	×5.5
M5×18.9	M130×28.1
M4×6	M100×8.9
×4.08	×6.1
×3.45	×5.1
×3.2	×4.8
M3×2.9	M75×4.3

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
S24×121	S610×180
×106	×158
S24×100	S610×149
×90	×134
×80	×119
S20×96	S510×143
×86	×128
S20×75	S510×112
×66	×98
S18×70	S460×104
×54.7	×81.4
S15×50	S380×74
×42.9	×64
S12×50	S310×74
×40.8	×60.7
S12×35	S310×52
×31.8	×47.3
S10×35	S250×52
×25.4	×37.8
S8×23	S200×34
×18.4	×27.4
S6×17.2	S150×25.7
×12.5	×18.6
S5×10	S130×15
S4×9.5	S100×14.1
×7.7	×11.5
S3×7.5	S75×11.2
×5.7	×8.5

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
HP14×117	HP360×174
×102	×152
×89	×132
×73	×108
HP12×84	HP310×125
×74	×110
×63	×93
×53	×79
HP10×57	HP250×85
×42	×62
HP8×36	HP200×53

Table 17–3

SI Equivalents of Standard U.S. Shape Profiles

Channels

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
C15×50	C380×74
×40	×60
×33.9	×50.4
C12×30	C310×45
×25	×37
×20.7	×30.8
C10×30	C250×45
×25	×37
×20	×30
×15.3	×22.8
C9×20	C230×30
×15	×22
×13.4	×19.9
C8×18.5	C200×27.9
×13.7	×20.5
×11.5	×17.1
C7×14.7	C180×22
×12.2	×18.2
×9.8	×14.6
C6×13	C150×19.3
×10.5	×15.6
×8.2	×12.2
C5×9	C130×13
×6.7	×10.4
C4×7.2	C100×10.8
×5.4	×8
×4.5	×6.7
C3×6	C75×8.9
×5	×7.4
×4.1	×6.1
×3.5	×5.2

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
MC18×58	MC460×86
×51.9	×77.2
×45.8	×68.2
×42.7	×63.5
MC13×50	MC330×74
×40	×60
×35	×52
×31.8	×47.3
MC12×50	MC310×74
×45	×67
×40	×60
×35	×52
×31	×46
MC12×10.6	MC310×15.8
MC10×41.1	MC250×61.2
×33.6	×50
×28.5	×42.4
MC10×25	MC250×37
×22	×33
MC10×8.4	MC250×12.5
×6.5	×9.7
MC9×25.4	MC230×37.8
×23.9	×35.6
MC8×22.8	MC200×33.9
×21.4	×31.8
MC8×20	MC200×29.8
×18.7	×27.8
MC8×8.5	MC200×12.6
MC7×22.7	MC180×33.8
×19.1	×28.4
MC6×18	MC150×26.8
×15.3	×22.8
MC6×16.3	MC150×24.3
×15.1	×22.5
MC6×12	MC150×17.9
MC6×7	MC150×10.4
×6.5	×9.7
MC4×13.8	MC100×20.5
MC3×7.1	MC75×10.6

Table 17–4

SI Equivalents of Standard U.S. Shape Profiles

Angles

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
L8×8×1 1/8	L203×203×28.6
×1	×25.4
×7/8	×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
L8×6×1	L203×152×25.4
×7/8	×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
×7/16	×11.1
L8×4×1	L203×102×25.4
×7/8	×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
×7/16	×11.1
L7×4×3/4	L178×102×19.0
×5/8	×15.9
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
L6×6×1	L152×152×25.4
×7/8	×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
L6×4×7/8	L152×102×22.2
×3/4	×19.0
×5/8	×15.9
×9/16	×14.3
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
L6×3 1/2×1/2	L152×89×12.7
×3/8	×9.5
×5/16	×7.9
L5×5×7/8	L127×127×22.2
×3/4	×19.0
×5/8	×15.9
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
L5×3 1/2×3/4	L127×89×19.0
×5/8	×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L5×3×1/2	L127×76×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L4×4×3/4	L102×102×19
×5/8	×15.9
×1/2	×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L4×3 1/2×1/2	L102×89×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L4×3×5/8	L102×76×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L3 1/2×3 1/2×1/2	L89×89×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L3 1/2×3×1/2	L89×76×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L3 1/2×2 1/2×1/2	L89×64×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
L3×3×1/2	L76×76×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
L3×2 1/2×1/2	L76×64×12.7
×7/16	×11.1
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8

Table 17–4 (continued)

SI Equivalents of Standard U.S. Shape Profiles

Angles

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	**mm × mm × mm**	**in. × in. × in.**	**mm × mm × mm**	**in. × in. × in.**	**mm × mm × mm**
L3×2×${}^{1}/_{2}$	L76×51×12.7	L2${}^{1}/_{2}$×2×${}^{3}/_{8}$	L64×51×9.5	L2×2×${}^{3}/_{8}$	L51×51×9.5
×${}^{3}/_{8}$	×9.5	×${}^{5}/_{16}$	×7.9	×${}^{5}/_{16}$	×7.9
×${}^{5}/_{16}$	×7.9	×${}^{1}/_{4}$	×6.4	×${}^{1}/_{4}$	×6.4
×${}^{1}/_{4}$	×6.4	×${}^{3}/_{16}$	×4.8	×${}^{3}/_{16}$	×4.8
×${}^{3}/_{16}$	×4.8			×${}^{1}/_{8}$	×3.2
		L2${}^{1}/_{2}$×1${}^{1}/_{2}$×${}^{1}/_{4}$	L64×38×6.4		
L2${}^{1}/_{2}$×2${}^{1}/_{2}$×${}^{1}/_{2}$	L64×64×12.7	×${}^{3}/_{16}$	×4.8		
×${}^{3}/_{8}$	×9.5				
×${}^{5}/_{16}$	×7.9				
×${}^{1}/_{4}$	×6.4				
×${}^{3}/_{16}$	×4.8				

Table 17–5

SI Equivalents of Standard U.S. Shape Profiles

WT Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m	in. × lb/ft	mm × kg/m
WT22×167.5	WT550×249.5	WT18×128	WT460×190.5	WT13.5×269.5	WT345×401
×145	×216.5	×116	×172.5	×184	×274
×131	×195	×105	×156.5	×168	×250
×115	×171.5	×97	×144.5	×153.5	×228.5
WT20×296.5	WT500×441.5	×91	×135.5	×140.5	×209.5
×251.5	×374	×85	×126.5	×129	×192
×215.5	×321	×80	×119	×117.5	×175
×198.5	×295.5	×75	×111.5	×108.5	×161.5
×186	×277	×67.5	×100.5	×97	×144.5
×181	×269.5	WT16.5×193.5	WT420×288	×89	×132.5
×162	×241.5	×177	×263.5	×80.5	×120
×148.5	×221.5	×159	×236.5	×73	×108.5
×138.5	×206	×145.5	×216.5	WT13.5×64.5	WT345×96
×124.5	×185.5	×131.5	×196	×57	×85
×107.5	×160.5	×120.5	×179.5	×51	×76
×99.5	×148	×110.5	×164.5	×47	×70
WT20×196	WT500×292	×100.5	×149.5	×42	×62.5
×165.5	×247	WT16.5×84.5	WT460×125.5	WT12×185	WT305×275.5
×163.5	×243	×76	×113	×167.5	×249
×147	×219	×70.5	×105	×153	×227.5
×139	×207.5	×65	×96.5	×139.5	×207.5
×132	×196.5	×59	×88	×125	×186
×117.5	×175	WT15×195.5	WT380×291	×114.5	×170.5
×105.5	×157	×178.5	×265.5	×103.5	×153.5
×91.5	×136	×163	×242	×96	×142.5
×83.5	×124.5	×146	×217	×88	×131
×74.5	×111	×130.5	×194.5	×81	×120.5
WT18×400	WT460×595.5	×117.5	×175	×73	×108.5
×326	×485	×105.5	×157	×65.5	×97.5
×264.5	×393.5	×95.5	×142	×58.5	×87
×243.5	×362.5	WT15×86.5	WT380×128.5	×52	×77.5
×220.5	×328	×74	×110	WT12×51.5	WT305×76.5
×197.5	×294	×66	×98	×47	×70
×180.5	×268.5	×62	×92.5	×42	×62.5
×165	×245.5	×58	×86.5	×38	×56.5
×151	×224.5	×54	×80.5	×34	×50.5
×141	×210	×49.5	×73.5	WT12×31	WT12×46
×131	×195	×45	×67	×27.5	×41
×123.5	×184				
×115.5	×172.5				

Table 17–5 (continued)

SI Equivalents of Standard U.S. Shape Profiles

WT Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
WT10.5×100.5	WT265×150
×91	×136
×83	×124
×73.5	×109.5
×66	×98
×61	×91
×55.5	×82.5
×50.5	×75
WT10.5×46.5	WT265×69
×41.5	×61.5
×36.5	×54.5
×34	×50.5
×31	×46
×27.5	×41
×24	×36
WT10.5×28.5	WT265×42.5
×25	×37
×22	×33
WT9×155.5	WT230×232
×141.5	×210.5
×129	×192
×117	×174.5
×105.5	×157.5
×96	×143
×87.5	×130
×79	×117.5
×71.5	×106.5
×65	×96.5
×59.5	×88.5
×53	×79
×48.5	×72
×43	×64
×38	×56.5
WT9×35.5	WT230×53
×32.5	×48.5
×30	×44.5
×27.5	×41
×25	×37
WT9×23	WT230×34
×20	×30
×17.5	×26
WT8×50	WT205×74.5
×44.5	×66
×38.5	×57
×33.5	×50
WT8×28.5	WT205×42.5
×25	×37.5
×22.5	×33.5
×20	×30
×18	×26.5
WT8×15.5	WT205×23.05
×13	×19.4
WT7×365	WT180×543
×332.5	×495
×302.5	×450
×275	×409
×250	×372
×227.5	×338.5
×213	×317
×199	×296
×185	×275.5
×171	×254.5
×155.5	×231.5
×141.5	×210.5
×128.5	×191
×116.5	×173.5
×105.5	×157
×96.5	×143.5
×88	×131
×79.5	×118.5
×72.5	×108
WT7×66	WT180×98
×60	×89.5
×54.5	×81
×49.5	×73.5
×45	×67
WT7×41	WT180×61
×37	×55
×34	×50.5
×30.5	×45.5
WT7×26.5	WT180×39.5
×24	×36
×21.5	×32
WT7×19	WT180×29
×17	×25.5
×15	×22.3
WT7×13	WT180×19.5
×11	×16.45
WT6×168	WT155×250
×152.5	×227
×139.5	×207.5
×126	×187.5
×115	×171
×105	×156.5
×95	×141.5
×85	×126.5
×76	×113
×68	×101
×60	×89.5
×53	×79
×48	×71.5
×43.5	×64.5
×39.5	×58.5
×36	×53.5
×32.5	×48.5
WT6×29	WT6×43
×26.5	×39.5
WT6×25	×37
×22.5	×33.5
×20	×30
WT6×17.5	WT6×26
×15	×22.25
×13	×19.35
WT6×11	WT6×16.35
×9.5	×14.15
×8	×11.9
×7	×10.5

Table 17–5 (continued)

SI Equivalents of Standard U.S. Shape Profiles

WT Shapes

Shape	SI Equivalent	Shape	SI Equivalent	Shape	SI Equivalent
in. × lb/ft	**mm × kg/m**	**in. × lb/ft**	**mm × kg/m**	**in. × lb/ft**	**mm × kg/m**
WT5×56	WT125×83.5	WT5×9.5	WT125×14.2	WT4×7.5	WT100×11.25
×50	×74.5	×8.5	×12.65	×6.5	×9.65
×44	×65.5	×7.5	×11.15	×5	×7.5
×38.5	×57.5	×6	×8.95	WT3×12.5	WT75×18.55
×34	×50.5	WT4×33.5	WT100×50	×10	×14.9
×30	×44.5	×29	×43	×7.5	×11.25
×27	×40	×24	×35.5	WT3×8	WT75×12
×24.5	×36.5	×20	×29.5	×6	×9
WT5×22.5	WT125×33.5	×17.5	×26	×4.5	×6.75
×19.5	×29	×15.5	×23.05	×4.25	×6.5
×16.5	×24.55	WT4×14	WT100×20.85	WT2.5×9.5	WT65×14.05
WT5×15	WT125×22.4	×12	×17.95	×8	×11.9
×13	×19.25	WT4×10.5	WT100×15.65	WT2×6.5	WT50×9.65
×11	×16.35	×9	×13.3		

Table 17–6

SI Equivalents of Standard U.S. Shape Profiles

MT and ST Shapes

Shape	SI Equivalent
in. × lb/ft	mm × kg/m
MT6.25×6.2	MT159×9.70
×5.8	×8.65
MT6×5.9	MT155×8.80
MT6×5.4	MT155×8.05
MT6×5	MT125×7.45
MT5×4.5	MT125×6.70
5×4	×5.95
MT5×3.75	MT125×5.60
MT4×3.25	MT100×4.85
×3.1	×4.25
MT3×2.2	MT75×3.3
×1.85	×2.75
MT2.5×9.45	MT65×14.1
MT2×3	MT50×4.45
×2.04	×3.05
×1.725	×2.55
×1.6	×2.4
MT1.5×1.45	MT37.5×2.15
ST12×60.5	ST305×90
×53	×79
ST12×50	ST305×75
×45	×67
×40	×60
ST10×48	ST254×72
×43	×64
ST10×37.5	ST254×56
×33	×49
ST9×35	ST230×52
×27.35	×41
ST7.5×25	ST190×37
×21.45	×32
ST6×25	ST152×37
×20.4	×30
ST6×17.5	ST152×26
×15.9	×24
ST5×17.5	ST127×26
×12.7	×19
ST4×11.5	ST102×17
×9.2	×14
ST3×8.6	ST76.2×13
×6.25	×9.3
ST2.5×5	ST63.5×7.5
ST2×4.75	ST50.8×7.1
×3.85	×5.7
ST1.5×3.75	ST38.1×5.6
×2.85	×4.25

Table 17–7

SI Equivalents of Standard U.S. Shape Profiles

Rectangular HSS

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
HSS20×12×5/8	HSS508×304.8×15.9	HSS14×6×5/8	HSS355.6×152.4×15.9
×1/2	×12.7	×1/2	×12.7
×3/8	×9.5	×3/8	×9.5
×5/16	×7.9	×5/16	×7.9
HSS20×8×5/8	HSS508×203.2×15.9	×1/4	×6.4
×1/2	×12.7	×3/16	×4.8
×3/8	×9.5	HSS14×4×5/8	HSS355.6×101.6×15.9
×5/16	×7.9	×1/2	×12.7
HSS20×4×1/2	HSS508×101.6×12.7	×3/8	×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
HSS18×6×5/8	HSS457.2×152.4×15.9	HSS12×10×1/2	HSS304.8×254×12.7
×1/2	×12.7	×3/8	×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	HSS12×8×5/8	HSS304.8×203.2×15.9
HSS16×12×5/8	HSS406.4×304.8×15.9	×1/2	×12.7
×1/2	×12.7	×3/8	×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
HSS16×8×5/8	HSS406.4×203.2×15.9	×3/16	×4.8
×1/2	×12.7	HSS12×6×5/8	HSS304.8×152.4×15.9
×3/8	×9.5	×1/2	×12.7
×5/16	×7.9	×3/8	×9.5
×1/4	×6.4	×5/16	×7.9
HSS16×4×5/8	HSS406.4×101.6×15.9	×1/4	×6.4
×1/2	×12.7	×3/16	×4.8
×3/8	×9.5	HSS12×4×5/8	HSS304.8×101.6×15.9
×5/16	×7.9	×1/2	×12.7
×1/4	×6.4	×3/8	×9.5
×3/16	×4.8	×5/16	×7.9
HSS14×10×5/8	HSS355.6×254×15.9	×1/4	×6.4
×1/2	×12.7	×3/16	×4.8
×3/8	×9.5	HSS12×3 1/2×3/8	HSS304.8×88.9×9.5
×5/16	×7.9	×5/16	×7.9
×1/4	×6.4	HSS12×3×5/16	HSS304.8×76.2×7.9
		×1/4	×6.4
		×3/16	×4.8

Table 17–7 (continued)
SI Equivalents of Standard U.S. Shape Profiles
Rectangular HSS

Shape	SI Equivalent
in. × in. × in.	mm × mm × mm
HSS12×2×3/8	HSS304.8×50.8×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×8×5/8	HSS254×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×6×5/8	HSS254×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×5×3/8	HSS254×127×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×4×5/8	HSS254×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS10×3 1/2×1/2	HSS254×88.9×4.8
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS10×3×3/8	HSS254×76.2×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS10×2×3/8	HSS254×50.8×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS9×7×5/8	HSS228.6×177.8×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×5×5/8	HSS228.6×127×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×3×1/2	HSS228.6×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS8×6×5/8	HSS203.2×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS8×4×5/8	HSS203.2×101.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS8×3×1/2	HSS203.2×76.2×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Table 17-7 (continued)

SI Equivalents of Standard U.S. Shape Profiles

Rectangular HSS

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
HSS8×2×3/8	HSS203.2×50.8×9.5	HSS6×3×1/2	HSS152.4×76.2×12.7
×5/16	×7.9	×3/8	×9.5
×1/4	×6.4	×5/16	×7.9
×3/16	×4.8	×1/4	×6.4
×1/8	×3.2	×3/16	×4.8
HSS7×5×1/2	HSS177.8×127×12.7	×1/8	×3.2
×3/8	×9.5	HSS6×2×3/8	HSS152.4×50.8×9.5
×5/16	×7.9	×5/16	×7.9
×1/4	×6.4	×1/4	×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS7×4×1/2	HSS177.8×101.6×12.7	HSS5×4×1/2	HSS127×101.6×12.7
×3/8	×9.5	×3/8	×9.5
×5/16	×7.9	×5/16	×7.9
×1/4	×6.4	×1/4	×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS7×3×1/2	HSS177.8×76.2×12.7	HSS5×3×1/2	HSS127×76.2×12.7
×3/8	×9.5	×3/8	×9.5
×5/16	×7.9	×5/16	×7.9
×1/4	×6.4	×1/4	×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS7×2×1/4	HSS177.8×50.8×6.4	HSS5×2 1/2×1/4	HSS127×63.5×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS6×5×1/2	HSS152.4×127×12.7	HSS5×2×3/8	HSS127×50.8×9.5
×3/8	×9.5	×5/16	×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
×3/16	×4.8	×1/8	×3.2
×1/8	×3.2	HSS4×3×3/8	HSS101.6×76.2×9.5
HSS6×4×1/2	HSS152.4×101.6×12.7	×5/16	×7.9
×3/8	×9.5	×1/4	×6.4
×5/16	×7.9	×3/16	×4.8
×1/4	×6.4	×1/8	×3.2
×3/16	×4.8		
×1/8	×3.2		

Table 17-7 (continued)

SI Equivalents of Standard U.S. Shape Profiles

Rectangular HSS

Shape	SI Equivalent	Shape	SI Equivalent
in. × in. × in.	mm × mm × mm	in. × in. × in.	mm × mm × mm
HSS4×2½×3/8	HSS101.6×63.5×9.5	HSS3×2×5/16	HSS76.2×50.8×7.9
×5/16	×7.9	×1/4	×6.4
×1/4	×6.4	×3/16	×4.8
×3/16	×4.8	×1/8	×3.2
×1/8	×3.2	HSS3×1½×1/4	HSS76.2×38.1×6.4
HSS4×2×3/8	HSS101.6×50.8×9.5	×3/16	×4.8
×5/16	×7.9	×1/8	×3.2
×1/4	×6.4	HSS3×1×3/16	HSS76.2×25.4×4.8
×3/16	×4.8	×1/8	×3.2
×1/8	×3.2	HSS2½×2×1/4	HSS63.5×50.8×6.4
HSS3½×2½×3/8	HSS88.9×63.5×9.5	×3/16	×4.8
×5/16	×7.9	×1/8	×3.2
×1/4	×6.4	HSS2½×1½×1/4	HSS63.5×38.1×6.4
×3/16	×4.8	×3/16	×4.8
×1/8	×3.2	×1/8	×3.2
HSS3½×2×1/4	HSS88.9×50.8×6.4	HSS2½×1×3/16	HSS63.5×25.4×4.8
×3/16	×4.8	×1/8	×3.2
×1/8	×3.2	HSS2¼×2×3/16	HSS57.2×50.8×4.8
HSS3½×1½×1/4	HSS88.9×38.1×6.4	×1/8	×3.2
×3/16	×4.8	HSS2×1½×3/16	HSS50.8×38.1×4.8
×1/8	×3.2	×1/8	×3.2
HSS3×2½×5/16	HSS76.2×63.5×7.9	HSS2×1×3/16	HSS50.8×25.4×4.8
×1/4	×6.4	×1/8	×3.2
×3/16	×4.8		
×1/8	×3.2		

Table 17–8

SI Equivalents of Standard U.S. Shape Profiles

Square HSS

Shape	SI Equivalent
in. × in.	mm × mm
HSS16×16×5/8	HSS406.4×406.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS14×14×5/8	HSS355.6×355.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
HSS12×12×5/8	HSS304.8×304.8×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS10×10×5/8	HSS254×254×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
HSS9×9×5/8	HSS228.6×228.6×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS8×8×5/8	HSS203.2×203.2×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS7×7×5/8	HSS177.8×177.8×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS6×6×5/8	HSS152.4×152.4×15.9
×1/2	×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5 1/2×5 1/2×3/8	HSS139.7×139.7×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS5×5×1/2	HSS127×127×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS4 1/2×4 1/2×1/2	HSS114.3×114.3×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2
HSS4×4×1/2	HSS101.6×101.6×12.7
×3/8	×9.5
×5/16	×7.9
×1/4	×6.4
×3/16	×4.8
×1/8	×3.2

Table 17-8 (continued)

SI Equivalents of Standard U.S. Shape Profiles

Square HSS

Shape	SI Equivalent
in. × in.	mm × mm
HSS3$^{1}/_{2}$×3$^{1}/_{2}$×$^{3}/_{8}$	HSS88.9×88.9×9.5
×$^{5}/_{16}$	×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS3×3×$^{3}/_{8}$	HSS76.2×76.2×9.5
×$^{5}/_{16}$	×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS2$^{1}/_{2}$×2$^{1}/_{2}$×$^{5}/_{16}$	HSS63.5×63.5×7.9
×$^{1}/_{4}$	×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS2$^{1}/_{4}$×2$^{1}/_{4}$×$^{1}/_{4}$	HSS57.2×57.2×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2
HSS2×2×$^{1}/_{4}$	HSS50.8×50.8×6.4
×$^{3}/_{16}$	×4.8
×$^{1}/_{8}$	×3.2

Table 17–9

SI Equivalents of Standard U.S. Shape Profiles

Round HSS and Pipe

Shape	SI Equivalent
in. × in.	mm × mm
HSS20.000×0.500	HSS508×12.7
×0.375	×9.5
HSS18.000×0.500	HSS457.2×12.7
×0.375	×9.5
HSS16.000×0.625	HSS406.4×15.9
×0.500	×12.7
×0.438	×11.1
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
HSS14.000×0.625	HSS355.6×15.9
×0.500	×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
HSS12.750×0.500	HSS323.9×12.7
×0.375	×9.5
×0.250	×6.4
HSS10.750×0.500	HSS273.1×12.7
×0.375	×9.5
×0.250	×6.4
HSS10.000×0.625	HSS254×15.9
×0.500	×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
HSS9.625×0.500	HSS244.5×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
HSS8.625×0.625	HSS219.1×15.9
×0.500	×12.7
×0.375	×9.5
×0.322	×8.2
×0.250	×6.4
×0.188	×4.8
HSS7.625×0.375	HSS193.7×9.5
×0.328	×8.3
HSS7.500×0.500	HSS190.5×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
HSS7.000×0.500	HSS177.8×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
×0.125	×3.2
HSS6.875×0.500	HSS174.6×12.7
×0.375	×9.5
×0.312	×7.9
×0.250	×6.4
×0.188	×4.8
HSS6.625×0.500	HSS168.3×12.7
×0.432	×11
×0.375	×9.5
×0.312	×7.9
×0.280	×7.1
×0.250	×6.4
×0.188	×4.8
×0.125	×3.2
HSS6.000×0.500	HSS152.4×12.7
×0.375	×9.5
×0.312	×7.9
×0.280	×7.1
×0.250	×6.4
×0.188	×4.8
×0.125	×3.2
HSS5.563×0.500	HSS141.3×12.7
×0.375	×9.5
×0.258	×6.6
×0.188	×4.8
×0.134	×3.4
HSS5.500×0.500	HSS139.7×12.7
×0.375	×9.5
×0.258	×6.6
HSS5.000×0.500	HSS127×12.7
×0.375	×9.5
×0.312	×7.9
×0.258	×6.6
×0.250	×6.4
×0.188	×4.8
×0.125	×3.2
HSS4.500×0.375	HSS114.3×9.5
×0.337	×8.6
×0.237	×6.0
×0.188	×4.8
×0.125	×3.2

Table 17–9 (continued)

SI Equivalents of Standard U.S. Shape Profiles

Round HSS and Pipe

Shape	SI Equivalent
in. × in.	mm × mm
HSS4.000×0.313	HSS101.6×8.0
×0.250	×6.4
×0.237	×6.0
×0.226	×5.7
×0.220	×5.6
×0.188	×4.8
×0.125	×3.2
HSS3.500×0.313	HSS88.9×8
×0.300	×7.6
×0.250	×6.4
×0.216	×5.5
×0.203	×5.2
×0.188	×4.8
×0.125	×3.2
HSS3.000×0.250	HSS76.2×6.4
×0.216	×5.5
×0.203	×5.2
×0.188	×4.8
×0.152	×3.9
×0.134	×3.4
×0.125	×3.2
HSS2.875×0.250	HSS73×6.4
×0.203	×5.2
×0.188	×4.8
×0.125	×3.2
HSS2.500×0.250	HSS63.5×6.4
×0.188	×4.8
×0.125	×3.2
HSS2.375×0.250	HSS60.3×6.4
×0.218	×5.5
×0.188	×4.8
×0.154	×3.9
×0.125	×3.2
HSS1.900×0.188	HSS48.3×4.8
×0.145	×3.7
×0.120	×3.0
HSS1.660×0.140	HSS42.2×3.6

Shape	SI Equivalent
in. × in.	mm × mm
PIPE ½ STD	PIPE 13 STD
PIPE ¾ STD	PIPE 19 STD
PIPE 1 STD	PIPE 25 STD
PIPE 1¼ STD	PIPE 32 STD
PIPE 1½ STD	PIPE 38 STD
PIPE 2 STD	PIPE 51 STD
PIPE 2½ STD	PIPE 64 STD
PIPE 3 STD	PIPE 75 STD
PIPE 3½ STD	PIPE 89 STD
PIPE 4 STD	PIPE 102 STD
PIPE 5 STD	PIPE 127 STD
PIPE 6 STD	PIPE 152 STD
PIPE 8 STD	PIPE 203 STD
PIPE 10 STD	PIPE 254 STD
PIPE 12 STD	PIPE 310 STD
PIPE ½ XS	PIPE 13 XS
PIPE ¾ XS	PIPE 19 XS
PIPE 1 XS	PIPE 25 XS
PIPE 1¼ XS	PIPE 32 XS
PIPE 1½ XS	PIPE 38 XS
PIPE 2 XS	PIPE 51 XS
PIPE 2½ XS	PIPE 64 XS
PIPE 3 XS	PIPE 75 XS
PIPE 3½ XS	PIPE 89 XS
PIPE 4 XS	PIPE 102 XS
PIPE 5 XS	PIPE 127 XS
PIPE 6 XS	PIPE 152 XS
PIPE 8 XS	PIPE 203 XS
PIPE 10 XS	PIPE 254 XS
PIPE 12 XS	PIPE 310 XS
PIPE 2 XXS	PIPE 51 XXS
PIPE 2½ XXS	PIPE 64 XXS
PIPE 3 XXS	PIPE 75 XXS
PIPE 4 XXS	PIPE 102 XXS
PIPE 5 XXS	PIPE 127 XXS
PIPE 6 XXS	PIPE 152 XXS
PIPE 8 XXS	PIPE 203 XXS

Table 17–10

Wire and Sheet Metal Gages

Equivalent thickness in decimals of an inch

Gage No.	U.S. Standard Gage for Uncoated Hot-& Cold-Rolled Sheets[b]	Galvanized Sheet Gage for Hot-Dipped Zinc-Coated Sheets[b]	USA Steel Wire Gage
7/0	–	–	.490
6/0	–	–	.462[a]
5/0	–	–	.430[a]
4/0	–	–	.394[a]
3/0	–	–	.362[a]
2/0	–	–	.331
1/0	–	–	.306
1	–	–	.283
2	–	–	.262[a]
3	.2391	–	.244[a]
4	.2242	–	.225[a]
5	.2092	–	.207
6	.1943	–	.192
7	.1793	–	.177
8	.1644	.1681	.162
9	.1495	.1532	.148[a]
10	.1345	.1382	.135
11	.1196	.1233	.120[a]
12	.1046	.1084	.106[a]
13	.0897	.0934	.092[a]
14	.0747	.0785	.080
15	.0673	.0710	.072
16	.0593	.0635	.062[a]
17	.0538	.0575	.054
18	.0478	.0516	.048[a]
19	.0418	.0456	.041
20	.0359	.0396	.035[a]
21	.0329	.0366	–
22	.0299	.0336	–
23	.0269	.0306	–
24	.0239	.0276	–
25	.0209	.0247	–
26	.0179	.0217	–
27	.0164	.0202	–
28	.0149	.0187	–
29	–	.0172	–
30	–	.0157	–

[a] Rounded value. The steel wire page has been taken from ASTM A510 "General Requirements for Wire Rods and Coarse Round Wire, Carbon Steel." Sizes originally quoted to four decimal equivalent places have been rounded to three decimal places in accordance with rounding procedures of ASTM "Recommended Practice" E29.

[b] The equivalent thicknesses are for information only. The product is commonly specified to decimal thickness (mils), not to gage number.

Table 17–11
Coefficients of Expansion

The coefficient of linear expansion (ε) is the change in length, per unit of length, for a change of one degree of temperature. The coefficient of surface expansion is approximately two times the linear coefficient, and the coefficient of volume expansion, for solids, is approximately three times the linear coefficient.

A bar, free to move, will increase in length with an increase in temperature and will decrease in length with a decrease in temperature. The change in length will be $\varepsilon t l$, where ε is the coefficient of linear expansion, t the change in temperature and l the length. If the ends of a bar are fixed, a change in temperature (t) will cause a change in the unit stress of $E\varepsilon t$, and in force of $AE\varepsilon t$, where A is the cross-sectional area of the bar and E the modulus of elasticity.

The following table gives the coefficient of linear expansion for 100°, or 100 times the value indicated above.

Example: A piece of medium steel is exactly 40 ft long at 60°F. Find the length at 90°F assuming the ends are free to move.

$$\text{change of length} = \varepsilon t l = \frac{.00065 \times 30 \times 40}{100} = .0078 \text{ ft}$$

The length at 90°F is 40.0078 ft

Example: A piece of medium carbon steel is exactly 40 ft long and the ends are fixed. If the temperature increases 30°F, what is the resulting change in the unit stress?

$$\text{change in unit stress} = E\varepsilon t = \frac{29{,}000 \times .00065 \times 30}{100} = 5.7 \text{ ksi}$$

COEFFICIENTS OF EXPANSION FOR 100 DEGREES = 100ε

	Linear Expansion			Linear Expansion	
Materials	**Celcius**	**Fahrenheit**	**Materials**	**Celcius**	**Fahrenheit**
METALS AND ALLOYS			STONE AND MASONRY		
Aluminum, wrought	.00231	.00128	Ashlar masonry	.00063	.00035
Brass	.00188	.00104	Brick masonry	.00061	.00034
Bronze	.00181	.00101	Cement, portland	.00126	.00070
Copper	.00168	.00093	Concrete	.00099	.00055
Iron, cast, gray	.00106	.00059	Granite	.00080	.00044
Iron, wrought	.00120	.00067	Limestone	.00076	.00042
Iron, wire	.00124	.00069	Marble	.00081	.00045
Lead	.00286	.00159	Plaster	.00166	.00092
Magnesium, various alloys	.0029	.0016	Rubble masonry	.00063	.00035
Nickel	.00126	.00070	Sandstone	.00097	.00054
Steel, mild	.00117	.00065	Slate	.00080	.00044
Steel, stainless, 18-8	.00178	.00099			
Zinc, rolled	.00311	.00173			
TIMBER			TIMBER		
Fir } parallel to fiber	.00037	.00021	Fir } perpendicular to fiber	.0058	.0032
Maple	.00064	.00036	Maple	.0048	.0027
Oak	.00049	.00027	Oak	.0054	.0030
Pine	.00054	.00030	Pine	.0034	.0019

EXPANSION OF WATER

Maximum Density = 1

°C	Volume	°C	Volume	°C	Volume	°C	Volume	°C	Volume	°C	Volume
0	1.000126	10	1.000257	30	1.004234	50	1.011877	70	1.022384	90	1.035829
4	1.000000	20	1.001732	40	1.007627	60	1.016954	80	1.029003	100	1.043116

Table 17–12

Densities of Common Materials

Substance	Weight lb per ft^3
ASHLAR, MASONRY	
Granite, syenite, gneiss	143 – 187
Limestone, marble	143 – 174
Sandstone, bluestone	131 – 150
MORTAR RUBBLE MASONRY	
Granite, syenite, gneiss	137 – 174
Limestone, marble	137 – 162
Sandstone, bluestone	125 – 137
DRY RUBBLE MASONRY	
Granite, syenite, gneiss	118 – 143
Limestone, marble	118 – 131
Sandstone, bluestone	112 – 118
BRICK MASONRY	
Pressed brick	137 – 143
Common brick	112 – 125
Soft brick	93.5 – 106
CONCRETE MASONRY	
Cement, stone, sand	137 – 150
Cement, slag, etc.	118 – 143
Cement, cinder, etc.	93.5 – 106
VARIOUS BUILDING MATERIALS	
Ashes, cinders	40.0 – 45.0
Cement, portland, loose	90.0
Cement, portland, set	168 – 199
Lime, gypsum, loose	53.0 – 64.0
Mortar, set	87.2 – 118
Slags, bank slag	67.0 – 72.0
Slags, bank screenings	98 – 117
Slags, machine slag	96.0
Slag, slag sand	49.0 – 55.0
EARTH, ETC., EXCAVATED	
Clay, dry	63.0
Clay, damp, plastic	110
Clay and gravel, dry	100
Earth, dry, loose	76.0
Earth, dry, packed	95.0
Earth, moist, loose	78.0
Earth, moist, packed	96.0
Earth, mud, flowing	108
Earth, mud, packed	115
Riprap, limestone	80.0 – 85.0
Riprap, sandstone	90.0
Riprap, shale	105
Sand, gravel, dry, loose	90.0 – 105
Sand, gravel, dry, packed	100 – 120
Sand, gravel, wet	118 – 120
EXCAVATIONS IN WATER	
Sand or gravel	60.0
Sand or gravel and clay	65.0
Clay	80.0
River mud	90.0
Soil	70.0
Stone riprap	65.0
MINERALS	
Asbestos	131 – 174
Barytes	280
Basalt	168 – 199
Bauxite	159
Borax	106 – 112
Chalk	112 – 162
Clay, marl	112 – 162
Dolomite	181
Feldspar, orthoclase	156 – 162
Gneiss, serpentine	150 – 168
Granite, syenite	156 – 193
Greenstone, trap	174 – 199
Gypsum, alabaster	143 – 174
Hornblende	187
Limestone, marble	156 – 174
Magnesite	187
Phosphate rock, apatite	199
Porphyry	162 – 181
Pumice, natural	23.1 – 56.1
Quartz, flint	156 – 174
Sandstone, bluestone	137 – 156
Shale, slate	168 – 181
Soapstone, talc	162 – 174
STONE, QUARRIED, PILED	
Basalt, granite, gneiss	96.0
Limestone, marble, quartz	95.0
Sandstone	82.0
Shale	92.0
Greenstone, hornblende	107
BITUMINOUS SUBSTANCES	
Asphaltum	68.5 – 93.5
Coal, anthracite	87.2 – 106
Coal, bituminous	74.8 – 93.5
Coal, lignite	68.5 – 87.2
Coal, peat, turf, dry	40.5 – 53
Coal, charcoal, pine	17.4 – 27.4
Coal, charcoal, oak	29.3 – 35.5
Coal, coke	62.3 – 87.2
Graphite	118 – 143
Paraffine	54.2 – 56.7
Petroleum	54.2
Petroleum, refined	49.2 – 51.1
Petroleum, benzine	45.5 – 46.7
Petroleum, gasoline	41.1 – 43
Pitch	66.7 – 71.6
Tar, bituminous	74.8
COAL AND COKE, PILED	
Coal, anthracite	47.0 – 58.0
Coal, bituminous, lignite	40.0 – 54.0

Table 17–12 (continued)

Densities of Common Materials

Substance	Weight lb per ft^3	Substance	Weight lb per ft^3
Coal, peat, turf	20.0 – 26.0	Starch	95.3
Coal charcoal	10.0 – 14.0	Sulphur	120 – 129
Coal coke	23.0 – 32.0	Wool	82.2
METALS, ALLOYS, ORES		**TIMBER, U.S. SEASONED**	
Aluminum, cast, hammered	159 – 171	Moisture content by weight:	
Brass, cast, rolled	523 – 542	Seasoned timber 15 to 20%	
Bronze, 7.9 to 14% Sn	461 – 554	Green timber up to 50%	
Bronze, aluminum	480	Ash, white, red	38.6 – 40.5
Copper, cast, rolled	548 – 561	Cedar, white, red	19.9 – 23.7
Copper ore, pyrites	255 – 268	Chestnut	41.1
Gold, cast, hammered	1200–1210	Cypress	29.9
Iron, cast, pig	449	Fir, Douglas spruce	31.8
Iron, wrought	473 – 492	Fir, eastern	24.9
Iron, speigel–eisen	467	Elm, white	44.9
Iron, ferro–silicon	417 – 455	Hemlock	26.2 – 32.4
Iron ore, hematite	324	Hickory	46.1 – 52.3
Iron ore, hematite in bank	160 – 180	Locust	45.5
Iron ore, hematite loose	130 – 160	Maple, hard	42.4
Iron ore, limonite	224 – 249	Maple, white	33.0
Iron ore, magnetite	305 – 324	Oak, chestnut	53.6
Iron slag	156 – 187	Oak, live	59.2
Lead	710	Oak, red, black	40.5
Lead ore, galena	455 – 473	Oak, white	46.1
Magnesium, alloys	108 – 114	Pine, Oregon	31.8
Manganese	449 – 498	Pine, red	29.9
Manganese, ore, pyrolusite	231 – 287	Pine, white	25.5
Mercury	847	Pine, yellow, long–leaf	43.6
Monel Metal	548 – 561	Pine, yellow, short–leaf	38.0
Nickel	554 – 573	Poplar	29.9
Platinum, cast, hammered	1310 – 1340	Redwood, California	26.2
Silver, cast, hammered	648 – 668	Spruce, white, black	24.9 – 28.7
Steel, rolled	490	Walnut, black	38.0
Tin, cast, hammered	449 – 467	Walnut, white	25.5
Tin ore, cassiterite	399 – 436		
Zinc, cast, rolled	430 – 449	**VARIOUS LIQUIDS**	
Zinc, ore, blende	243 – 262	Alcohol, 100%	49.2
		Acids, muriatic 40%	74.8
VARIOUS SOLIDS		Acids, nitric 91%	93.5
Cereals, oats, bulk	32.0	Acids, sulphuric 87%	112
Cereals, barley, bulk	39.0	Lye, soda 66%	106
Cereals, corn, rye, bulk	48.0	Oils, vegetable	56.7 – 58.6
Cereals, wheat, bulk	48.0	Oils, mineral, lubricants	56.1 – 57.9
Hay and Straw, bales	20.0	Water, 4°C max. density	62.3
Cotton, Flax, Hemp	91.6 – 93.5	Water, 100°C	59.7
Fats	56.1 – 60.4	Water, ice	54.8 – 57.3
Flour, loose	24.9 – 31.2	Water, sea water	63.5 – 64.2
Flour, pressed	43.6 – 49.8		
Glass, common	150 – 162	**GASES**	
Glass, plate or crown	153 – 169	Air, 0°C 760 mm	0.0871
Glass, crystal	181 – 187	Ammonia	0.0478
Leather	53.6 – 63.5	Carbon dioxide	0.123
Paper	43.6 – 71.6	Carbon monoxide	0.078
Potatoes, piled	42.0	Gas, illuminating	0.028–0.036
Rubber, caoutchouc	57.3 – 59.8	Gas, natural	0.038–0.039
Rubber goods	62.3 – 125	Hydrogen	0.00559
Sald, granulated, piled	48.0	Nitrogen	0.0784
Saltpeter	67.0	Oxygen	0.0892

Table 17–13

Weights of Building Materials

Materials	Weight lb per sq ft	Materials	Weight lb per sq ft
CEILINGS		**PARTITIONS**	
Channel suspended system	1	Clay Tile	
Lathing and plastering	See Partitions	3 in.	17
Acoustical fiber tile	1	4 in.	18
		6 in.	28
		8 in.	34
		10 in.	40
FLOORS		Gypsum Block	
Steel Deck	See Manufacturer	2 in.	9½
		3 in.	10½
Concrete-Reinforced 1 in.		4 in.	12½
Stone	12½	5 in.	14
Slag	11½	6 in.	18½
Lightweight	6 to 10	Wood Studs 2×4	
		12–16 in. o.c.	2
Concrete-Plain 1 in.		Steel partitions	4
Stone	12	Plaster 1 inch	
Slag	11	Cement	10
Lightweight	3 to 9	Gypsum	5
		Lathing	
Fills 1 inch		Metal	½
Gypsum	6	Gypsum Board ½-in.	2
Sand	8		
Cinders	4		
Finishes			
Terrazzo 1 in.	13		
Ceramic or Quarry Tile ¾-in.	10		
Linoleum ¼-in.	1	**WALLS**	
Mastic ¾-in.	9	Brick	
Hardwood ⅞-in.	4	4 in.	40
Softwood ¾-in.	2½	8 in.	80
		12 in.	120
		Hollow Concrete Block (Heavy Aggregate)	
ROOFS		4 in.	30
Copper or tin	1	6 in.	43
Corrugated steel	See Manufactuer	8 in.	55
3-ply ready roofing	1	12½-in.	80
3-ply felt and gravel	5½	Hollow Concrete Block	
5-ply felt and gravel	6	(Light Aggregate)	
		4 in.	21
Shingles		6 in.	30
Wood	2	8 in.	38
Asphalt	3	12 in.	55
Clay tile	9 to 14	Clay tile (Load Bearing)	
Slate ¼	10	4 in.	25
		6 in.	30
Sheathing		8 in.	33
Wood ¾-in.	3	12 in.	45
Gypsum 1 in.	4	Stone 4 in.	55
		Glass Block 4 in.	18
Insulation 1 in.		Window, Glass, Frame, & Sash	8
Loose	½	Curtain Walls	See Manufacturer
Poured	2	Structural Glass 1 in.	15
Rigid	1½	Corrugated Cement Asbestos ¼-in.	3

For weights of other materials used in building construction, see Table 17-12.

Table 17-14
Weights and Measures
United States System

LINEAR MEASURE

Inches	*Feet*	*Yards*	*Rods*	*Furlongs*	*Miles*
1.0	.08333	.02778	.0050505	.00012626	.00001578
12.0	1.0	.33333	.0606061	.00151515	.00018939
36.0	3.0	1.0	.1818182	.00454545	.00056818
198.0	16.5	5.5	1.0	.025	.003125
7,920.0	660.0	220.0	40.0	1.0	.125
63,360.0	5,280.0	1,760.0	320.0	8.0	1.0

SQUARE AND LAND MEASURE

Sq. Inches	*Square Feet*	*Square Yards*	*Square Rods*	*Acres*	*Sq. Miles*
1.0	.006944	.000772			
144.0	1.0	.111111			
1,296.0	9.0	1.0	.03306	.000207	
39,204.0	272.25	30.25	1.0	.00625	.0000098
	43,560.0	4,840.0	160.0	1.0	.0015625
		3,097,600.0	102,400.0	640.0	1.0

AVOIRDUPOIS WEIGHTS

Grains	*Drams*	*Ounces*	*Pounds*	*Tons*
1.0	.03657	.002286	.000143	.0000000714
27.34375	1.0	.0625	.003906	.00000195
437.5	16.0	1.0	.0625	.00003125
7,000.0	256.0	16.0	1.0	.0005
14,000,000.0	512,000.0	32,000.0	2,000.0	1.0

DRY MEASURE

Pints	*Quarts*	*Pecks*	*Cubic Feet*	*Bushels*
1.0	.5	.0625	.01945	.01563
2.0	1.0	.125	.03891	.03125
16.0	8.0	1.0	.31112	.25
51.42627	25.71314	3.21414	1.0	.80354
64.0	32.0	4.0	1.2445	1.0

LIQUID MEASURE

Gills	*Pints*	*Quarts*	*U.S. Gallons*	*Cubic Feet*
1.0	.25	.125	.03125	.00418
4.0	1.0	.5	.125	.01671
8.0	2.0	1.0	.250	.03342
32.0	8.0	4.0	1.0	.1337
			7.48052	1.0

SI UNITS FOR STRUCTURAL STEEL DESIGN

Although there are seven metric base units in the SI system, only four are currently used by AISC in structural steel design. These base units are listed in Table 17-15.

Table 17–15. Base SI Units for Steel Design

Quantity	Unit	Symbol
Length	meter	m
mass	kilogram	kg
time	second	s
temperature	celcius	°C

Similarly, of the numerous decimal prefixes included in the SI system, only three are used in steel design; see Table 17-16.

Table 17–16. SI Prefixes for Steel Design

Prefix	Symbol	Order of Magnitude	Expression
mega	M	10^6	1,000,000 (one million)
kilo	k	10^3	1,000 (one thousand)
milli	m	10^{-3}	0.001 (one thousandth)

In addition, three derived units are applicable to the present conversion. They are shown in Table 17-17.

Table 17–17. Derived SI Units for Steel Design

Quantity	Name	Symbol	Expression
force	newton	N	$N = kg \times m/s^2$
stress	pascal	Pa	$Pa = N/m^2$
energy	joule	J	$J = N \times m$

Although specified in SI, the pascal is not universally accepted as the unit of stress. Because section properties are expressed in millimeters, it is more convenient to express stress in newtons per square millimeter ($1\ N/mm^2 = 1$ MPa). This is the practice followed in recent international structural design standards. It should be noted that the joule, as the unit of energy, is used to express energy absorption requirements for impact tests. Moments are expressed in terms of N×m.

A summary of the conversion factors relating traditional U.S. units of measurement to the corresponding SI units is given in Table 17-18.

Table 17–18. Summary of SI Conversion Factors

Multiply	by:	to obtain:
inch (in.)	25.4	millimeters (mm)
foot (ft)	305	millimeters (mm)
pound-mass (lb)	0.454	kilogram (kg)
pound-force (lbf)	4.448	newton (N)
ksi	6.895	N/mm^2
ft-lbf	1.356	joule (J)
psf	47.88	N / m^2
plf	14.59	N / m

Note that fractions resulting from metric conversion should be rounded to whole millimeters. Common fractions of inches and their metric equivalents are in Table 17–19.

Table 17–19. SI Equivalents of Fractions of an Inch

Fraction, in.	Exact conversion, mm	Rounded to: (mm)
1/16	1.5875	2
1/8	3.175	3
3/16	4.7625	5
1/4	6.35	6
5/16	7.9375	8
3/8	9.525	10
7/16	11.1125	11
1/2	12.7	13
5/8	15.875	16
3/4	19.05	19
7/8	22.225	22
1	25.4	25

Bolt diameters are taken directly from the ASTM Specifications A325M and A490M rather than converting the diameters of SI bolts dimensioned in inches, since metric bolts are of different physical sizes. The metric bolt designations are in Table 17–20.

Table 17–20. SI Bolt Designation

Designation	Diameter, mm	Diameter, in.
M16	16	0.63
M20	20	0.79
M22	22	0.87
M24	24	0.94
M27	27	1.06
M30	30	1.18
M36	36	1.42

The yield strengths of structural steels are taken from the metric ASTM Specifications. It should be noted that the yield points are slightly different from the traditional values. See Table 17–21. The modulus of elasticity of steel E is taken as 200,000 N/mm^2. The shear modulus of elasticity of steel G is 77,000 N/mm^2.

Table 17–21. SI Steel Yield Stresses

ASTM Designation	Yield stress, N/mm^2	Yield stress, ksi
A36M	250	36.26
A572M Gr. 345 A588M	345	50.04
A852M	485	70.34
A514M	690	100.07

Table 17-22

Weights and Measures International System of Units (SI)[a]

(Metric practice)

BASE UNITS

Quantity	*Unit*	*Symbol*
length	meter	m
mass	kilogram	kg
time	second	s
electric current	ampere	A
thermodynamic temperature	kelvin	K
amount of substance	mole	mol
luminous intensity	candela	cd

SUPPLEMENTARY UNITS

Quantity	*Unit*	*Symbol*
plane angle	radian	rad
solid angle	steradian	sr

DERIVED UNITS (WITH SPECIAL NAMES)

Quantity	*Unit*	*Symbol*	*Formula*
force	newton	N	$kg\text{-}m/s^2$
pressure, stress	pascal	Pa	N/m^2
energy, work, quantity of heat	joule	J	N-m
power	watt	W	J/s

DERIVED UNITS (WITHOUT SPECIAL NAMES)

Quantity	*Unit*	*Formula*
area	square meter	m^2
volume	cubic meter	m^3
velocity	meter per second	m/s
acceleration	meter per second squared	m/s^2
specific volume	cubic meter per kilogram	m^3/kg
density	kilogram per cubic meter	kg/m^3

SI PREFIXES

Multiplication Factor	*Prefix*	*Symbol*
$1\ 000\ 000\ 000\ 000\ 000\ 000 = 10^{18}$	exa	E
$1\ 000\ 000\ 000\ 000\ 000 = 10^{15}$	peta	P
$1\ 000\ 000\ 000\ 000 = 10^{12}$	tera	T
$1\ 000\ 000\ 000 = 10^{9}$	giga	G
$1\ 000\ 000 = 10^{6}$	mega	M
$1\ 000 = 10^{3}$	kilo	k
$100 = 10^{2}$	hecto[b]	h
$10 = 10^{1}$	deka[b]	da
$0.1 = 10^{-1}$	deci[b]	d
$0.01 = 10^{-2}$	centi[b]	c
$0.001 = 10^{-3}$	milli	m
$0.000\ 001 = 10^{-6}$	micro	μ
$0.000\ 000\ 001 = 10^{-9}$	nano	n
$0.000\ 000\ 000\ 001 = 10^{-12}$	pico	p
$0.000\ 000\ 000\ 000\ 001 = 10^{-15}$	femto	f
$0.000\ 000\ 000\ 000\ 000\ 001 = 10^{-18}$	atto	a

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[a] Refer to ASTM E380 for more complete information on SI.
[b] Use is not recommended.

Table 17–23

SI Conversion Factors[a]

Quantity	Multiply	by	to obtain	
Length	inch	25.400	millimeter	mm
	foot	0.305	meter	m
	yard	0.914	meter	m
	mile (U.S. Statute)	1.609	kilometer	km
	millimeter	39.370×10^{-3}	inch	in
	meter	3.281	foot	ft
	meter	1.094	yard	yd
	kilometer	0.621	mile	mi
Area	square inch	0.645×10^{3}	square millimeter	mm^2
	square foot	0.093	square meter	m^2
	square yard	0.836	square meter	m^2
	square mile (U.S. Statute)	2.590	square kilometer	km^2
	acre	4.047×10^{3}	square meter	m^2
	acre	0.405	hectare	
	square millimeter	1.550×10^{-3}	square inch	in^2
	square meter	10.764	square foot	ft^2
	square meter	1.196	square yard	yd^2
	square kilometer	0.386	square mile	mi^2
	square meter	0.247×10^{-3}	acre	
	hectare	2.471	acre	
Volume	cubic inch	16.387×10^{3}	cubic millimeter	mm^3
	cubic foot	28.317×10^{-3}	cubic meter	m^3
	cubic yard	0.765	cubic meter	m^3
	gallon (U.S. liquid)	3.785	liter	*l*
	quart (U.S. liquid)	0.946	liter	*l*
	cubic millimeter	61.024×10^{-6}	cubic inch	in^3
	cubic meter	35.315	cubic foot	ft^3
	cubic meter	1.308	cubic yard	yd^3
	liter	0.264	gallon (U.S. liquid)	gal
	liter	1.057	quart (U.S. liquid)	qt
Mass	ounce (avoirdupois)	28.350	gram	g
	pound (avoirdupois)	0.454	kilogram	kg
	short ton	0.907×10^{3}	kilogram	kg
	gram	35.274×10^{-3}	ounce (avoirdupois)	oz av
	kilogram	2.205	pound (avoirdupois)	lb av
	kilogram	1.102×10^{-3}	short ton	

[a] Refer to ASTM E380 for more complete information on SI.
The conversion factors tabulated herein have been rounded.

Table 17–23 (continued)

SI Conversion Factors[a]

Quantity	Multiply	by	to obtain	
Force	[c]ounce-force	0.278	[c]newton	N
	[c]pound-force	4.448	[c]newton	N
	[c]newton	3.597	[c]ounce-force	
	[c]newton	0.225	[c]pound-force	lbf
Bending Moment	[c]pound-force-inch	0.113	[c]newton-meter	N-m
	[c]pound-force-foot	1.356	[c]newton-meter	N-m
	[c]newton-meter	8.851	[c]pound-force-inch	lbf-in
	[c]newton-meter	0.738	[c]pound-force-foot	lbf-ft
Pressure, Stress	[c]pound-force per square inch	6.895	[c]kilopascal	kPa
	[c]foot of water (39.2 F)	2.989	[c]kilopascal	kPa
	[c]inch of mercury (32 F)	3.386	[c]kilopascal	kPa
	[c]kilopascal	0.145	[c]pound-force per [c] square inch	lbf/in^2
	[c]kilopascal	0.335	[c]foot of water (39.2 F)	
	[c]kilopascal	0.295	[c]inch of mercury (32 F)	
Energy, Work, Heat	[c]foot-pound-force	1.356	[c]joule	J
	[b]British thermal unit	1.055×10^{3}	[c]joule	J
	[b]calorie	4.187	[c]joule	J
	[c]kilowatt hour	3.600×10^{6}	[c]joule	J
	[c]joule	0.738	[c]foot-pound-force	ft-lbf
	[c]joule	0.948×10^{-3}	[b]British thermal unit	Btu
	[c]joule	0.239	[b]calorie	
	[c]joule	0.278×10^{-6}	[c]kilowatt hour	kW-h
Power	[c]foot-pound-force/second	1.356	[c]watt	W
	[b]British thermal unit per hour	0.293	[c]watt	W
	[c]horsepower (550 ft lbf/s)	0.746	[c]kilowatt	kW
	[c]watt	0.738	[c]foot-pound-force/ [c] second	ft-lbf/s
	[c]watt	3.412	[b]British thermal unit [c] per hour	Btu/h
	kilowatt	1.341	[c]horsepower [c] (550 ft-lbf/s)	hp
Angle	[c]degree	17.453×10^{-3}	[c]radian	rad
	[c]radian	57.296	[c]degree	
Temperature	[c]degree Fahrenheit	$t^{\circ}C = (t^{\circ}F - 32)/1.8$	[c]degree Celsius	
	[c]degree Celsius	$t^{\circ}F = 1.8 \times t^{\circ}C + 32$	[c]degree Fahrenheit	

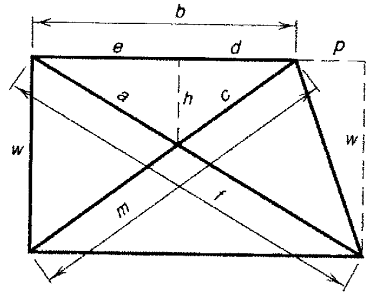
[a] Refer to ASTM E380 for more complete information on SI.

[b] International Table.

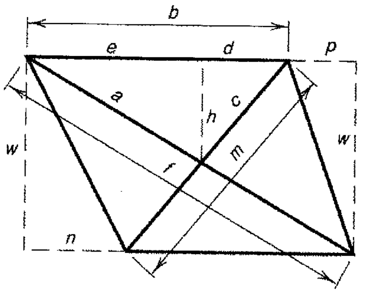
[c] The conversion factors tabulated herein have been rounded.

Table 17–24

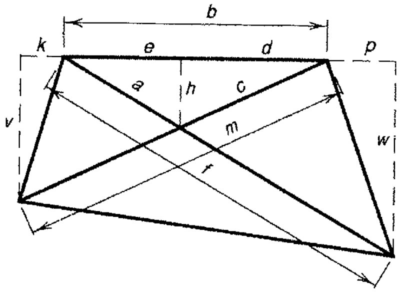
Bracing Formulas



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bw	m	$\sqrt{b^2+w^2}$
bp	d	$b^2 \div (2b+p)$
bp	e	$b(b+p) \div (2b+p)$
bfp	a	$bf \div (2b+p)$
bmp	c	$bm \div (2b+p)$
bpw	h	$bw \div (2b+p)$
afw	h	$aw \div f$
cmw	h	$cw \div m$



Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bnw	m	$\sqrt{(b-n)^2+w^2}$
bnp	d	$b(b-n) \div (2b+p-n)$
bnp	e	$b(b+p) \div (2b+p-n)$
bfnp	a	$bf \div (2b+p-n)$
bmnp	c	$bm \div (2b+p-n)$
bnpw	h	$bw \div (2b+p-n)$
afw	h	$aw \div f$
cmw	h	$cw \div m$



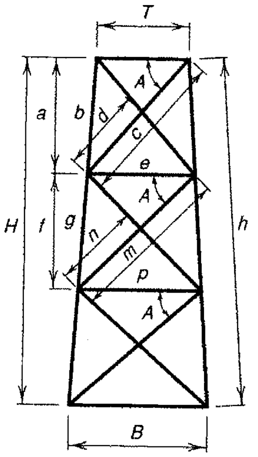
Given	To Find	Formula
bpw	f	$\sqrt{(b+p)^2+w^2}$
bkv	m	$\sqrt{(b+k)^2+v^2}$
bkpvw	d	$bw(b+k) \div [v(b+p)+w(b+k)]$
bkpvw	e	$bv(b+p) \div [v(b+p)+w(b+k)]$
bfkpvw	a	$fbv \div [v(b+p)+w(b+k)]$
bkmpvw	c	$bmw \div [v(b+p)+w(b+k)]$
bkpvw	h	$bvw \div [v(b+p)+w(b+k)]$
afw	h	$aw \div f$
cmv	h	$cv \div m$

PARALLEL BRACING

$k = (\log B - \log T) \div$ no. of panels. Constant k plus the logarithm of any line equals the log of the corresponding line in the next panel below.

$a = TH \div (T + e + p)$
$b = Th \div (T + e + p)$

$c = \sqrt{(1/2\,T + 1/2\,e)^2 + a^2}$
$d = ce \div (T + e)$



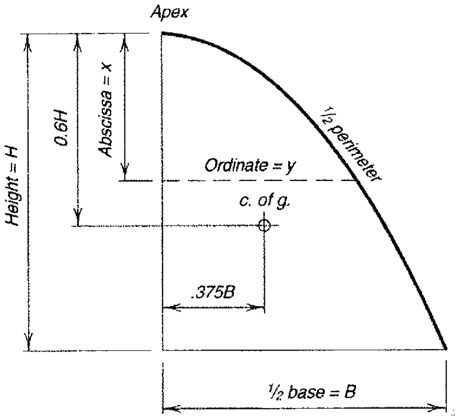
$\log e = k + \log T$
$\log f = k + \log a$
$\log g = k + \log b$
$\log m = k + \log c$
$\log n = k + \log d$
$\log p = k + \log e$

The above method can be used for any number of panels. In the formulas for "a" and "b" the sum in parenthesis, which in the case shown is $(T + e + p)$, is always composed of all the horizontal distances except the base.

Table 17–25

Properties of Parabola and Ellipse

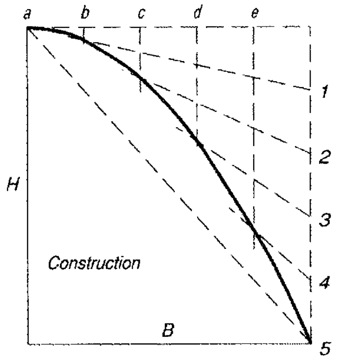
PARABOLA



Parameter $P = \frac{B^2}{H}$ Area = ⅔HB

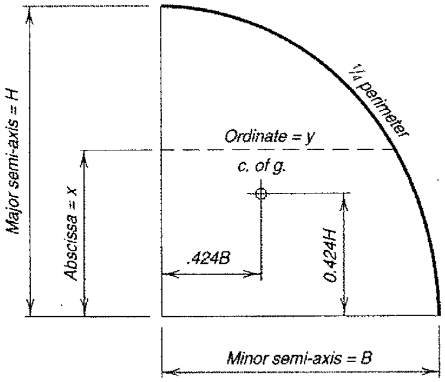
$x = \frac{y^2}{P}$

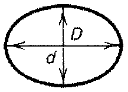
$y = \sqrt{xP}$



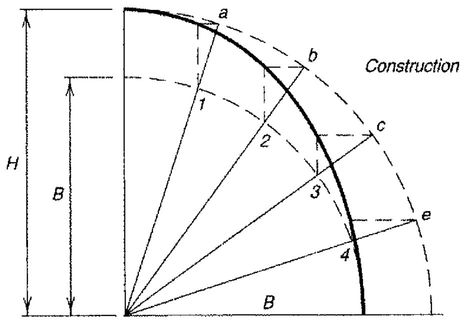
ELLIPSE

$(x^2 \div H^2) + (y^2 \div B^2) = 1$

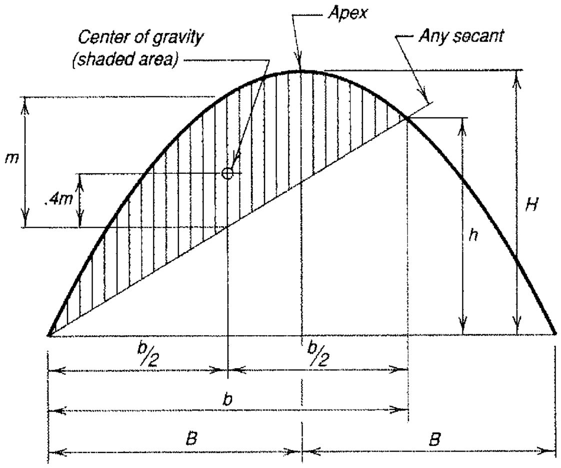




Area = .7854Dd



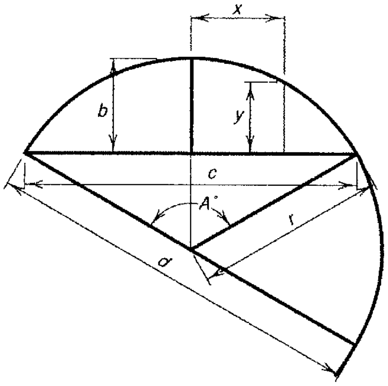
AREA BETWEEN PARABOLIC CURVE AND SECANT



Length b may vary from 0 to 2B

Table 17-26

Properties of the Circle



Circumference = 6.28378 r = 3.14159d
Diameter = 0.31831 circumference
Area = 3.14159r^2

$$\text{Arc } a = \frac{\pi r A^\circ}{180^\circ} = 0.017453 r A^\circ$$

$$\text{Angle } A^\circ = \frac{180^\circ a}{\pi r} = 57.29578 \frac{a}{r}$$

$$\text{Angle } A^\circ = 2 \sin^{-1}(c/2r)$$

$$\text{Angle } A^\circ = 4 \tan^{-1}(2b/c)$$

$$\text{Radius } r = \frac{4b^2 + c^2}{8b}$$

$$\text{Chord } c = 2\sqrt{2br - b^2} = 2r \sin \frac{A}{2}$$

$$\text{Rise } b = r - \frac{1}{2}\sqrt{4r^2 - c^2} = \frac{c}{2} \tan \frac{A}{4}$$

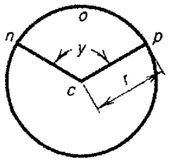
$$= 2r \sin^2 \frac{A}{4} = r + y - \sqrt{r^2 - x^2}$$

$$y = b - r + \sqrt{r^2 - x^2}$$

$$x = \sqrt{r^2 - (r + y - b)^2}$$

Diameter of circle of equal periphery as square = 1.27324 side of square
Side of square of equal periphery as circle = 0.78540 diameter of circle
Diameter of circle circumscribed about square = 1.41421 side of square
Side of square inscribed in circle = 0.70711 diameter of circle

CIRCULAR SECTOR

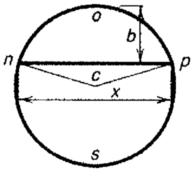


r = radius of circle y = angle ncp in degrees
Area of Sector $ncpo$ = ½ (length of arc $nop \times r$)

$$= \text{Area of Circle} \times \frac{y}{360}$$

$$= 0.0087266 \times r^2 \times y$$

CIRCULAR SEGMENT



r = radius of circle x = chord b = rise
Area of Segment nop = Area of Sector ncpo − Area of triangle ncp

$$= \frac{(\text{Length of arc } nop \times r) - x(r - b)}{2}$$

r = radius of circle y = angle ncp in degrees
Area of Sector $ncpo$ = ½ (length of arc $nop \times r$)

$$= \text{Area of Circle} \times \frac{y}{360}$$

$$= 0.0087266 \times r^2 \times y$$

Table 17–27
Properties of Geometric Sections

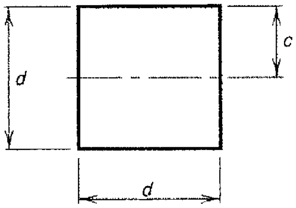
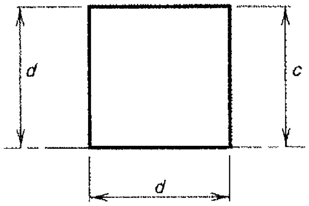
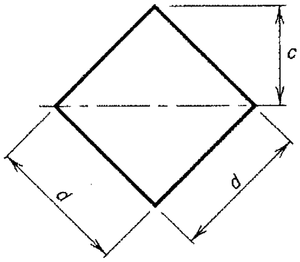
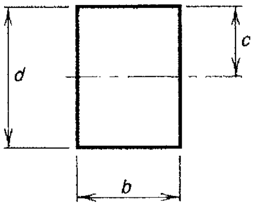
SQUARE *Axis of moments through center*  	$A = d^2$ $c = \frac{d}{2}$ $I = \frac{d^4}{12}$ $S = \frac{d^3}{6}$ $r = \frac{d}{\sqrt{12}} = .288675\ d$ $Z = \frac{d^3}{4}$
SQUARE *Axis of moments on base*  	$A = d^2$ $c = d$ $I = \frac{d^4}{3}$ $S = \frac{d^3}{3}$ $r = \frac{d}{\sqrt{3}} = .577350\ d$
SQUARE *Axis of moments on diagonal*  	$A = d^2$ $c = \frac{d}{\sqrt{2}} = .707107\ d$ $I = \frac{d^4}{12}$ $S = \frac{d^3}{6\sqrt{2}} = .117851\ d^3$ $r = \frac{d}{\sqrt{12}} = .288675\ d$ $Z = \frac{2c^3}{3} = \frac{d^3}{3\sqrt{2}} = .235702\ d^3$
RECTANGLE *Axis of moments through center*  	$A = bd$ $c = \frac{d}{2}$ $I = \frac{bd^3}{12}$ $S = \frac{bd^2}{6}$ $r = \frac{d}{\sqrt{12}} = .288675\ d$ $Z = \frac{bd^2}{4}$

Table 17–27 (continued)
Properties of Geometric Sections

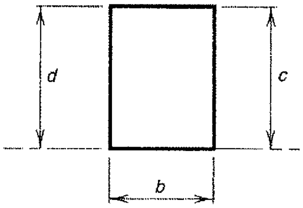
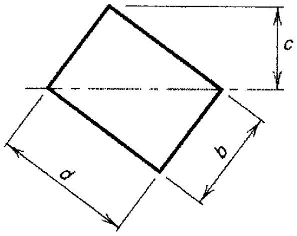
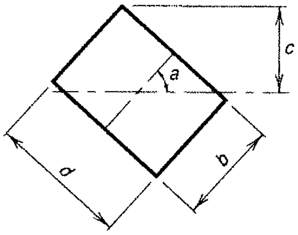
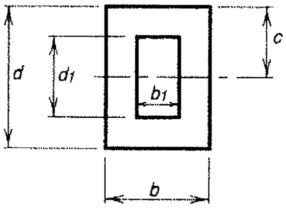
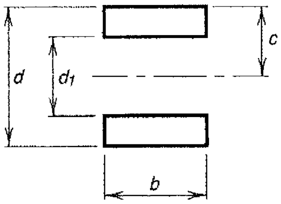
RECTANGLE *Axis of moments on base*  	$A = bd$ $c = d$ $I = \frac{bd^3}{3}$ $S = \frac{bd^2}{3}$ $r = \frac{d}{\sqrt{3}} = .577350\,d$
RECTANGLE *Axis of moments on diagonal*  	$A = bd$ $c = \frac{bd}{\sqrt{b^2 + d^2}}$ $I = \frac{b^3d^3}{6(b^2 + d^2)}$ $S = \frac{b^2d^2}{6\sqrt{b^2 + d^2}}$ $r = \frac{bd}{\sqrt{6(b^2 + d^2)}}$
RECTANGLE *Axis of moments any line* *through center of gravity*  	$A = bd$ $c = \frac{b \sin a + d \cos a}{2}$ $I = \frac{bd\,(b^2 \sin^2 a + d^2 \cos^2 a)}{12}$ $S = \frac{bd\,(b^2 \sin^2 a + d^2 \cos^2 a)}{6\,(b \sin a + d \cos a)}$ $r = \sqrt{\frac{b^2 \sin^2 a + d^2 \cos^2 a}{12}}$
HOLLOW RECTANGLE *Axis of moments through center*  	$A = bd - b_1d_1$ $c = \frac{d}{2}$ $I = \frac{bd^3 - b_1d_1^3}{12}$ $S = \frac{bd^3 - b_1d_1^3}{6d}$ $r = \sqrt{\frac{bd^3 - b_1d_1^3}{12A}}$ $Z = \frac{bd^2}{4} - \frac{b_1d_1^2}{4}$

Table 17–27 (continued)

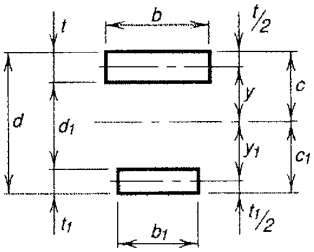
Properties of Geometric Sections

EQUAL RECTANGLES
Axis of moments through center of gravity



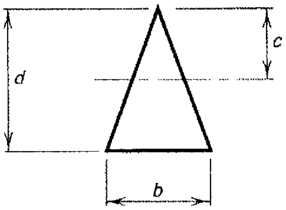
$$A = b(d - d_1)$$
$$c = \frac{d}{2}$$
$$I = \frac{b(d^3 - d_1^3)}{12}$$
$$S = \frac{b(d^3 - d_1^3)}{6d}$$
$$r = \sqrt{\frac{d^3 - d_1^3}{12(d - d_1)}}$$
$$Z = \frac{b}{4}(d^2 - d_1^2)$$

UNEQUAL RECTANGLES
Axis of moments through center of gravity



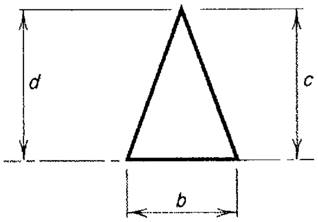
$$A = bt + b_1t_1$$
$$c = \frac{\frac{1}{2}bt^2 + b_1t_1(d - \frac{1}{2}t_1)}{A}$$
$$I = \frac{bt^3}{12} + bty^2 + \frac{b_1t_1^3}{12} + b_1t_1y_1^2$$
$$S = \frac{I}{c} \qquad S_1 = \frac{I}{c_1}$$
$$r = \sqrt{\frac{I}{A}}$$
$$Z = bty + b_1t_1y_1$$

TRIANGLE
Axis of moments through center of gravity



$$A = \frac{bd}{2}$$
$$c = \frac{2d}{3}$$
$$I = \frac{bd^3}{36}$$
$$S = \frac{bd^2}{24}$$
$$r = \frac{d}{\sqrt{18}} = .235702\ d$$

TRIANGLE
Axis of moments on base



$$A = \frac{bd}{2}$$
$$c = d$$
$$I = \frac{bd^3}{12}$$
$$S = \frac{bd^2}{12}$$
$$r = \frac{d}{\sqrt{6}} = .408248\ d$$

Table 17–27 (continued)
Properties of Geometric Sections

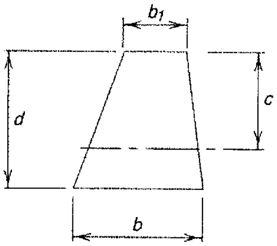
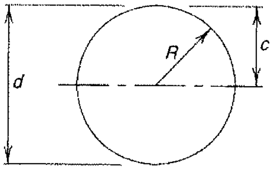
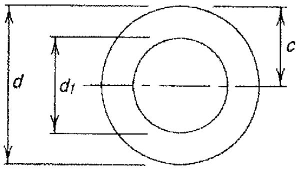
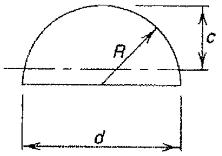
TRAPEZOID *Axis of moments through center of gravity*  	$A = \dfrac{d(b+b_1)}{2}$ $c = \dfrac{d(2b+b_1)}{3(b+b_1)}$ $I = \dfrac{d^3(b^2+4bb_1+b_1^2)}{36(b+b_1)}$ $S = \dfrac{d^2(b^2+4bb_1+b_1^2)}{12(2b+b_1)}$ $r = \dfrac{d}{6(b+b_1)}\sqrt{2(b^2+4bb_1+b_1^2)}$
CIRCLE *Axis of moments through center*  	$A = \dfrac{\pi d^2}{4} = \pi R^2 = .785398\ d^2 = 3.141593\ R^2$ $c = \dfrac{d}{2} = R$ $I = \dfrac{\pi d^4}{64} = \dfrac{\pi R^4}{4} = .049087\ d^4 = .785398\ R^4$ $S = \dfrac{\pi d^3}{32} = \dfrac{\pi R^3}{4} = .098175\ d^3 = .785398\ R^3$ $r = \dfrac{d}{4} = \dfrac{R}{2}$ $Z = \dfrac{d^3}{6}$
HOLLOW CIRCLE *Axis of moments through center*  	$A = \dfrac{\pi(d^2-d_1^2)}{4} = .785398\ (d^2-d_1^2)$ $c = \dfrac{d}{2}$ $I = \dfrac{\pi(d^4-d_1^4)}{64} = .049087\ (d^4-d_1^4)$ $S = \dfrac{\pi(d^4-d_1^4)}{32d} = .098175\ \dfrac{d^4-d_1^4}{d}$ $r = \dfrac{\sqrt{d^2+d_1^2}}{4}$ $Z = \dfrac{d^3}{6} - \dfrac{d_1^3}{6}$
HALF CIRCLE *Axis of moments through center of gravity*  	$A = \dfrac{\pi R^2}{2} = 1.570796\ R^2$ $c = R\left(1 - \dfrac{4}{3\pi}\right) = .575587\ R$ $I = R^4\left(\dfrac{\pi}{8} - \dfrac{8}{9\pi}\right) = .109757\ R^4$ $S = \dfrac{R^3}{24}\,\dfrac{(9\pi^2-64)}{(3\pi-4)} = .190687\ R^3$ $r = R\,\dfrac{\sqrt{9\pi^2-64}}{6\pi} = .264336\ R$

Table 17–27 (continued)
Properties of Geometric Sections

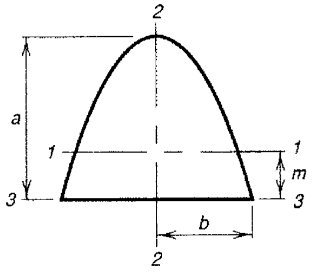
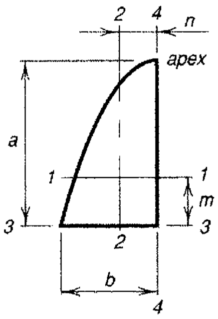
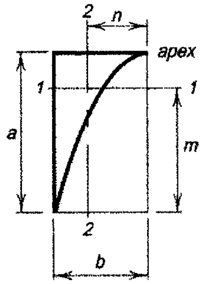
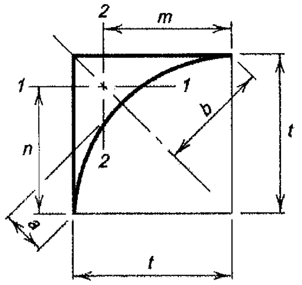
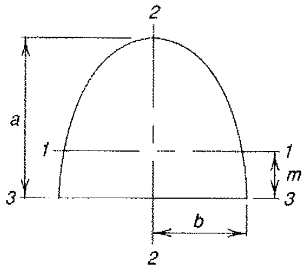
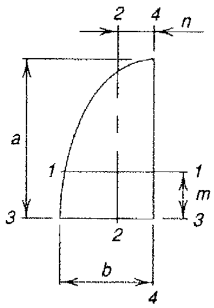
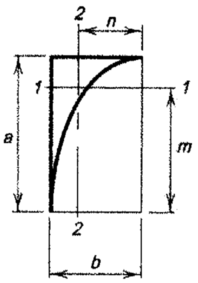
PARABOLA  	$A = \frac{4}{3}ab$ $m = \frac{2}{5}a$ $I_1 = \frac{16}{175}a^3b$ $I_2 = \frac{4}{15}ab^3$ $I_3 = \frac{32}{105}a^3b$
HALF PARABOLA  	$A = \frac{2}{3}ab$ $m = \frac{2}{5}a$ $n = \frac{3}{8}b$ $I_1 = \frac{8}{175}a^3b$ $I_2 = \frac{19}{480}ab^3$ $I_3 = \frac{16}{105}a^3b$ $I_4 = \frac{2}{15}ab^3$
COMPLEMENT OF HALF PARABOLA  	$A = \frac{1}{3}ab$ $m = \frac{7}{10}a$ $n = \frac{3}{4}b$ $I_1 = \frac{37}{2{,}100}a^3b$ $I_2 = \frac{1}{80}ab^3$
PARABOLIC FILLET IN RIGHT ANGLE  	$a = \frac{t}{2\sqrt{2}}$ $b = \frac{t}{\sqrt{2}}$ $A = \frac{1}{6}t^2$ $m = n = \frac{4}{5}t$ $I_1 = I_2 = \frac{11}{2{,}100}t^4$

Table 17–27 (continued)
Properties of Geometric Sections

*HALF ELLIPSE  	$A = \frac{1}{2}\pi ab$ $m = \frac{4a}{3\pi}$ $I_1 = a^3 b\left(\frac{\pi}{8} - \frac{8}{9\pi}\right)$ $I_2 = \frac{1}{8}\pi ab^3$ $I_3 = \frac{1}{8}\pi a^3 b$
*QUARTER ELLIPSE  	$A = \frac{1}{4}\pi ab$ $m = \frac{4a}{3\pi}$ $n = \frac{4b}{3\pi}$ $I_1 = a^3 b\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$ $I_2 = ab^3\left(\frac{\pi}{16} - \frac{4}{9\pi}\right)$ $I_3 = \frac{1}{16}\pi a^3 b$ $I_4 = \pi ab^3$
*ELLIPTIC COMPLEMENT  	$A = ab\left(1 - \frac{\pi}{4}\right)$ $m = \frac{a}{6\left(1 - \frac{\pi}{4}\right)}$ $n = \frac{b}{6\left(1 - \frac{\pi}{4}\right)}$ $I_1 = a^3 b\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36\left(1 - \frac{\pi}{4}\right)}\right)$ $I_2 = ab^3\left(\frac{1}{3} - \frac{\pi}{16} - \frac{1}{36\left(1 - \frac{\pi}{4}\right)}\right)$

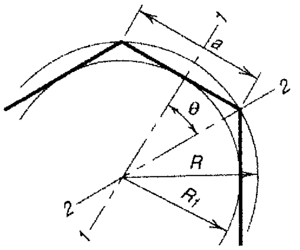
*To obtain properties of half circle, quarter circle, and circular complement, substitute $a = b = R$.

Table 17–27 (continued)

Properties of Geometric Sections

REGULAR POLYGON

Axis of moments through center



n = Number of sides

$$\theta = \frac{180^\circ}{n}$$

$$a = 2\sqrt{R^2 - R_1^2}$$

$$R = \frac{a}{2\sin\theta}$$

$$R_1 = \frac{a}{2\tan\theta}$$

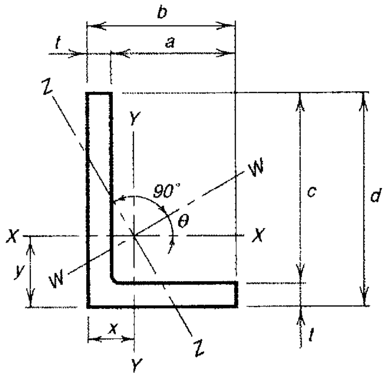
$$A = \frac{1}{4}na^2\cot\theta = \frac{1}{2}nR^2\sin 2\theta = nR_1^2\tan\theta$$

$$I_1 = I_2 = \frac{A(6R^2 - a^2)}{24} = \frac{A(12R_1^2 + a^2)}{48}$$

$$r_1 = r_2 = \sqrt{\frac{6R^2 - a^2}{24}} = \sqrt{\frac{12R_1^2 + a^2}{48}}$$

ANGLE

Axis of moments through center of gravity



$$\tan 2\theta = \frac{2K}{I_Y - I_X}$$

$$A = t(b + c) \quad x = \frac{b^2 + ct}{2(b + c)} \quad y = \frac{d^2 + at}{2(b + c)}$$

K = Product of Inertia about X X and Y Y

$$= \pm\frac{abcdt}{4(b + c)}$$

$$I_X = \frac{1}{3}\left(t(d - y)^3 + by^3 - a(y - t)^3\right)$$

$$I_Y = \frac{1}{3}\left(t(b - x)^3 + dx^3 - c(x - t)^3\right)$$

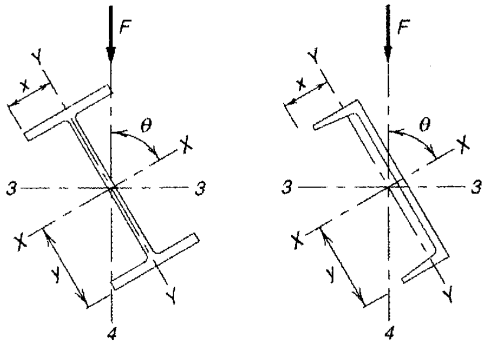
$$I_Z = I_X\sin^2\theta + I_Y\cos^2\theta + K\sin 2\theta$$

$$I_W = I_X\cos^2\theta + I_Y\sin^2\theta - K\sin 2\theta$$

K is negative when heel of angle, with respect to center of gravity, is in 1st or 3rd quadrant, positive when in 2nd or 4th quadrant.

BEAMS AND CHANNELS

Transverse force oblique through center of gravity



$$I_3 = I_X\sin^2\theta + I_Y\cos^2\theta$$

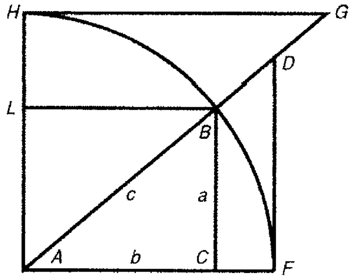
$$I_4 = I_X\cos^2\theta + I_Y\sin^2\theta$$

$$f_b = M\left(\frac{y}{I_X}\sin\theta + \frac{x}{I_Y}\cos\theta\right)$$

where M is bending moment due to force F.

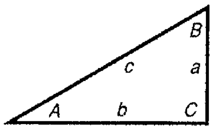
Table 17-28
Trigonometric Formulas

TRIGNOMETRIC FUNCTIONS



$$\text{Radius } AF = 1$$
$$= \sin^2 A + \cos^2 A = \sin A \operatorname{cosec} A$$
$$= \cos A \sec A = \tan A \cot A$$
$$\sin A = \frac{\cos A}{\cot A} = \frac{1}{\operatorname{cosec} A} = \cos A \tan A = \sqrt{1-\cos^2 A} = BC$$
$$\cos A = \frac{\sin A}{\tan A} = \frac{1}{\sec A} = \sin A \cot A = \sqrt{1-\sin^2 A} = AC$$
$$\tan A = \frac{\sin A}{\cos A} = \frac{1}{\cot A} = \sin A \sec A = FD$$
$$\cot A = \frac{\cos A}{\sin A} = \frac{1}{\tan A} = \cos A \operatorname{cosec} A = HG$$
$$\sec A = \frac{\tan A}{\sin A} = \frac{1}{\cos A} = AD$$
$$\operatorname{cosec} A = \frac{\cot A}{\cos A} = \frac{1}{\sin A} = AG$$

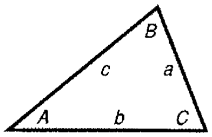
RIGHT ANGLED TRIANGLES



$$a^2 = c^2 - b^2$$
$$b^2 = c^2 - a^2$$
$$c^2 = a^2 + b^2$$

Known	Required A	Required B	Required a	Required b	Required c	Required Area
a, b	$\tan A = \frac{a}{b}$	$\tan B = \frac{b}{a}$			$\sqrt{a^2+b^2}$	$\frac{ab}{2}$
a, c	$\sin A = \frac{a}{c}$	$\cos B = \frac{a}{c}$		$\sqrt{c^2-a^2}$		$\frac{a\sqrt{c^2-a^2}}{2}$
A, a		$90° - A$		$a \cot A$	$\frac{a}{\sin A}$	$\frac{a^2 \cot A}{2}$
A, b		$90° - A$	$b \tan A$		$\frac{b}{\cos A}$	$\frac{b^2 \tan A}{2}$
A, c		$90° - A$	$c \sin A$	$c \cos A$		$\frac{c^2 \sin 2A}{4}$

OBLIQUE ANGLED TRIANGLES



$$s = \frac{a+b+c}{2}$$
$$K = \sqrt{\frac{(s-a)(s-b)(s-c)}{s}}$$

$$a^2 = b^2 + c^2 - 2bc \cos A$$
$$b^2 = a^2 + c^2 - 2ac \cos B$$
$$c^2 = a^2 + b^2 - 2ab \cos C$$

Known	Required A	Required B	Required C	Required b	Required c	Required Area
a, b, c	$\tan \frac{1}{2} A = \frac{K}{s-a}$	$\tan \frac{1}{2} B = \frac{K}{s-b}$	$\tan \frac{1}{2} C = \frac{K}{s-c}$			$\sqrt{s(s-a)(s-b)(s-c)}$
a, A, B			$180° - (A+B)$	$\frac{a \sin B}{\sin A}$	$\frac{a \sin C}{\sin A}$	
a, b, A		$\sin B = \frac{b \sin A}{a}$			$\frac{b \sin C}{\sin B}$	
a, b, C	$\tan A = \frac{a \sin C}{b - a \cos C}$				$\sqrt{a^2+b^2-2ab \cos C}$	$\frac{ab \sin C}{2}$

GENERAL NOMENCLATURE

The following definitions apply, as these variables are used in this Manual. Additional nomenclature used in both the Manual and the Specification can be found in the AISC *Specification for Structural Steel Buildings*, in Part 16 of this Manual.

A	Area of directly connected elements, in.^{2}
A	Horizontal distance from end panel point to mid-span of a truss, ft
A	Minimum side dimension for square or rectangular beveled washer, in.
A_b	Required transverse force from an adjacent bay, kips.
A_{cp}	Projected surface area of concrete cone surrounding headed anchor rods, in.^{2}
A_f	Area of flange, in.^{2}
A_{fe}	Effective tension flange area, in.^{2}
A_{gt}	Gross area subject to tension, in.^{2}
B	Available tensile strength per bolt subjected to prying action, kips.
B	Horizontal distance from mid-span of a truss to a given panel point, ft
B	Base plate width, in.
BF	A factor that can be used to calculate the flexural strength for unbraced length L_b between L_p and L_r
C	Required mid-span camber, in.
C	Width across points of square or hex bolt head or nut, or maximum diameter of countersunk bolt head, in.
C	Coefficient for eccentrically loaded bolt and weld groups
C_{Tot}	Sum of compressive forces in a composite beam, kips
C_c	Beam reaction coefficient
C_{conc}	Effective concrete flange force for a composite beam, kips
C_{stl}	Compressive force in steel in a composite beam, kips
C_1	Loading constant used in deflection calculations
C_1	Clearance for tightening, in.
C_1	Electrode coefficient for relative strength of electrodes where, for E70 electrodes, $C_1 = 1.00$
C_2	Clearance for entering, in.
C_3	Clearance for fillet based on one standard hardened washer, in.
C'	Coefficient for eccentrically loaded bolt groups subjected to moment only
CG	Center of gravity
D	Offset from the base line at a panel point of a truss, in.
D	Number of sixteenths-of-an-inch in the weld size
E	Earthquake load
E	Minimum edge distance for clipped washer, in.
E	Minimum effective throat thickness for partial-joint-penetration groove weld, in.
ENA	Elastic neutral axis
F	Width across flats of bolt head, in.
F	Clearance for tightening staggered bolts, in.
F_p	Nominal bearing stress on fastener, ksi

F_{yb}	F_y of a beam, ksi
F_{yc}	F_y of a column, ksi
F_{yc}	F_y of a cap plate, ksi
F_{yf}	Specified minimum yield stress of the flange, ksi
G	Ratio of the total column stiffness framing into a joint to that of the stiffening members framing into the same joint
H	Horizontal force, kips
H	Height of bolt head or nut, in.
H	Theoretical thread height, in.
H_b	Required shear force on a beam to gusset connection, kips
H_c	Required axial force on a column go gusset connection, kips
H_1	Height of bolt head, in.
H_2	Maximum bolt shank extension based on one standard hardened washer, in.
I_{LB}	Lower bound moment of inertia for composite section, in.^{4}
I_c	Moment of inertia of column section about axis perpendicular to plane of buckling, in.^{4}
I_g	Moment of inertia of girder about axis perpendicular to plane of buckling, in.^{4}
I_p	Moment of inertia of primary member, in.^{4}
I_p	Polar moment of inertia of bolt and weld groups ($I_p = I_x + I_y$), in.^{4} per in.^{2}
I_{st}	Moment of inertia of a transverse stiffener, in.^{4}
I_x	Moment of inertia of bolt and weld groups about x-axis, in.^{4} per in.^{2}
I_y	Moment of inertia of bolt and weld groups about y-axis, in.^{4} per in.^{2}
I_{yc}	Moment of inertia about y-axis referred to compression flange, or if reverse curvature bending referred to smaller flange, in.^{4}
IC	Instantaneous center of rotation
ID	Nominal inside diameter of flat circular washer, in.
K	Minimum root diameter of threaded fastener, in.
K_{dep}	Fillet depth, $(k - t_f)$, in.
L	Length of connection in the direction of loading, in.
L	Live load due to occupancy and moveable equipment
L_c	Unsupported length of a column section, ft
L_e	Edge distance, in.
L_{eh}	Horizontal edge distance, in.
L_{ev}	Vertical edge distance, in.
L_g	Unsupported length of a girder or other restraining member, ft
L_h	Hook length for hooked anchor rods, in.
L_p'	Limiting laterally unbraced length for the maximum design flexural strength for noncompact shapes, uniform moment case ($C_b = 1.0$), in. or ft, as indicated
L_r	Roof live load
M	Beam bending moment, kip-in. or kip-ft, as indicated
M_{LL}	Beam moment due to live load, kip-in. or kip-ft, as indicated
M_{cr}	Elastic buckling moment, kip-in. or kip-ft, as indicated
M_p'	Maximum available flexural strength for noncompact shapes, when $L_b \le L_p'$, kip-in. or kip-ft, as indicated
M_{pa}	Plastic bending moment modified by axial load ratio, kip-in.
M_r	Limiting buckling moment, M_{cr}, when $\lambda = \lambda_r$ and $C_b = 1.0$, kip-in. or kip-ft, as indicated

M_u	Required flexural strength using LRFD load combinations, kip-in. or kip-ft, as indicated
N	Length of base plate, in.
N_b	Number of bolts in a joint
N_r	Number of shear stud connectors in one rib at a beam intersection.
N_r	Required length of bearing, in.
OD	Nominal outside diameter of flat circular washer, in.
P	Concentrated load, kips
P	Required axial force, kips
P	Bolt stagger, in.
P_a	Required concentrated beam load using ASD load combinations, kips
P_a	Required axial strength (tension or compression) using ASD load combinations, kips
P_{af}	Required beam flange force, tensile or compressive, using ASD load combinations, kips
P_e	Elastic (Euler) buckling load, kips
P_{fb}	Resistance to flange local bending per AISC Specification Equation J10-1 (used to check need for column web stiffeners), kips
P_u	Required concentrated beam load using LRFD load combinations, kips
P_{uf}	Factored beam flange force, tensile or compressive, using LRFD load combinations, kips
P_{wb}	Resistance to web compression buckling per AISC Specification Equation J10-8 (used to check need for column web stiffening), kips
P_{wi}	A factor consisting of terms from the second portion of AISC Specification Equation J10-2 (used in a column web stiffener check for web local yielding), kips/in.
P_{wo}	A factor consisting of the first portion of AISC Specification Equation J10-2 (used in a column web stiffener check for web local yielding), kips
PNA	Plastic neutral axis
R	Nominal reaction, kips
R	Required end reaction, kips
$R_{a\,st}$	Required strength for transverse stiffener (force delivered to stiffener) using ASD load combinations, kips
$R_{u\,st}$	Required strength for transverse stiffener (force delivered to stiffener) using LRFD load combinations, kips
R_v	Web shear strength, kips
R_w	Effective nominal strength of a concentrically loaded weld group, kips
R_1	Beam bearing constant, see Part 9.
R_2	Beam bearing constant, see Part 9.
R_3	Beam bearing constant, see Part 9.
R_4	Beam bearing constant, see Part 9.
R_5	Beam bearing constant, see Part 9.
R_6	Beam bearing constant, see Part 9.
S	Spacing, in. or ft, as indicated
S	Groove depth for partial-joint-penetration groove welds, in.
S_{net}	Net elastic section modulus, in.^{3}
T	Distance between web toes of fillets at top and at bottom of web, in. = $d - 2k$
T	Tension force due to service loads, kips

T	Thickness of flat circular washer or mean thickness of square or rectangular beveled washer, in.
T_{avail}	Available tensile strength, kips
T_{stl}	Tensile force in steel in a composite beam, kips
T_{Tot}	Sum of tensile forces in a composite beam, kips
V	Shear force, kips
V_a	Required shear strength using ASD load combinations, kips
V_b	Shear force component, kips
V_b	Required shear force on a beam to gusset connection, kips
V_u	Required shear strength using LRFD load combinations, kips
W	Wind load
W	Uniformly distributed load, kips
W	Weight, lbs or kips, as indicated
W	Width across flats of nut, in.
W_a	Total factored uniformly distributed load using ASD load combinations, kips
W_c	Uniform load constant for beams, kip-ft
W_u	Total factored uniformly distributed load using LRFD load combinations, kips
Y_{ENA}	Distance from bottom of steel beam to elastic neutral axis, in.
Y_{con}	Distance from top of steel beam to top of concrete, in.
Y_1	Distance from top of steel beam to the plastic neutral axis, in.
Y_2	Distance from top of steel beam to the concrete flange force in a composite beam, in.
Z_e	Effective plastic section modulus, in.^{3}
Z_{net}	Net plastic section modulus, in.^{3}
a	Effective concrete flange thickness of a composite beam, in.
a	Coefficient for eccentrically loaded weld group
a	Distance from bolt centerline to edge of fitting subjected to prying action, but not greater than 1.25b, in.
a	Distance from an HSS centroid to the end of an attached member, in.
a	Distance from the weld line to the first row of bolts in a single plate connection, in.
a'	Weld length, in.
b	Effective concrete flange width in a composite beam, in.
b	Minimum shelf dimension for deposition of fillet weld, in.
b	Distance from bolt centerline to face of fitting subjected to prying action, in.
b_{eff}	Effective width, in.
b_x	Coefficient for strong axis bending related to combined axial and bending strength calculations (see Part 6)
b_y	Coefficient for weak axis bending related to combined axial and bending strength calculations (see Part 6)
c	Distance from the neutral axis to the extreme fiber of the cross section, in.
c	Radial distance from the center of gravity to the portion of the weld group most remote from the center of gravity, in.
c	Cope length, in.
d_c	Cope depth, in.
d_{ct}	Top-flange cope depth, in.
d_{cb}	Bottom-flange cope depth, in.
d_h	Hole diameter, in.

d_m	Moment arm between resultant tensile and compressive forces due to a moment or eccentric force, in.
d_w	Diameter of a part in contact with the inner surface of an HSS, in.
d_z	Overall panel-zone depth, in.
e	Eccentricity, in.
e	Base of natural logarithms = 2.71828...
e_o	Horizontal distance from the outer edge of a channel web to its shear center, in.
f	Computed compressive stress in the stiffened element, ksi
f	Plate buckling model adjustment factor for beams coped at top flange only
f_b	Maximum bending stress, ksi
f_d	Adjustment factor for beams coped at both flanges
f_{un}	Required normal stress, ksi
f_{uv}	Required shear stress, ksi
g	Transverse center-to-center spacing (gage) between fastener gage lines, in.
g	Acceleration due to gravity = 32.2 ft/sec^2 = 386 in./sec^2
h_o	Distance between flange centroids, in.
h_r	Nominal rib height, in.
h_o	Remaining web depth of coped beam, in.
k	Plate buckling coefficient for beams coped at top flange only
k_1	Distance from web center line to flange toe of fillet, in.
kip	1,000 pounds
ksi	kips/in.^{2}
l	Span length, in.
l	Length of weld, in.
l	Characteristic length of weld group, in.
l_i	Distance of the ith bolt from the center of gravity, in.
l_o	Distance from center of gravity (CG) to instantaneous center of rotation (IC) of bolt or weld group, in.
m	Cantilever dimension for base plate, in.
n	Number of shear connectors between point of maximum positive moment and the point of zero moment to each side
n	Number of bolts in a vertical row
n	Cantilever dimension for base plate, in.
n'	Number of bolts above the neutral axis (in tension)
p	Length of supporting flange parallel to stem or leg of hanger tributary to each bolt in determining prying action, in.
p	Coefficient for axial compression related to combined axial and bending strength calculations (see Part 6)
q	Horizontal shear, kips/in.
q	Additional tension per bolt resulting from prying action produced by deformation of the connected parts, kips/bolt
r_a	Required shear strength per bolt using ASD load combinations, kips/bolt
r_{at}	Required tensile strength per bolt or per inch of weld using ASD load combinations (force per bolt or per inch of weld due to a tensile force), kips/bolt
r_{av}	Required shear strength per bolt or per inch of weld using ASD load combinations (force per bolt or per inch of weld due to a shear force), kips/bolt
r_m	Radius of gyration of steel shape, pipe, or tubing in composite columns.

r_m	Required shear force on the bolt most remote from the center of gravity, due to moment, kips
r_m	Shear per inch of weld due to moment, kips/in.
r_n	Nominal strength per bolt, kips
r_p	Required shear strength per bolt due to a concentric force, kips/bolt
r_u	Required shear strength per bolt using LRFD load combinations, kips/bolt
r_{ut}	Required tensile strength per bolt or per inch of weld using LRFD load combinations (force per bolt or per inch of weld due to a tensile force), kips/bolt
r_{uv}	Required shear strength per bolt or per inch of weld using LRFD load combinations (force per bolt or per inch of weld due to a shear force), kips/bolt
r_x, r_y	Radius of gyration about x and y axes respectively, in.
r_{yc}	Radius of gyration about y axis referred to compression flange, or if reverse curvature bending, referred to smaller flange, in.
$\bar{r}_o$	Polar radius of gyration about the shear center, in.
t	Change in temperature, degrees Fahrenheit or Celsius, as indicated
t_b	Thickness of beam flange or connection plate delivering concentrated force, in.
t_c	Flange or angle thickness required to develop design tensile strength of bolts with no prying action, in.
t_c	Thickness of cap plate, in.
t_{des}	Design thickness of an HSS wall, in.
t_f	Lesser connection element thickness, in.
t_{nom}	Nominal thickness of an HSS wall, in.
t_r	Coefficient for tension rupture related to combined axial and bending strength calculations (see Part 6)
t_s	Tee stem thickness, in.
t_{wb}	Beam web thickness, in.
t_{wc}	Column web thickness, in.
t_y	Coefficient for tension yielding related to combined axial and bending strength calculations (see Part 6)
w	Uniformly distributed load per unit of length, kips/in.
w	Plate width; distance between welds, in.
x	Horizontal distance, in.
x	Horizontal distance from the support to the location of applied bearing force, in.
$\bar{x}$	Horizontal distance from the outer edge of a channel web to its centroid, in.
x_p	Horizontal distance from the designated edge of member to its plastic neutral axis, in.
x_o	Horizontal distance, in.
y	Moment arm between centroid of tensile forces and compressive forces, in.
y_p	Vertical distance from the designated edge of member to its plastic neutral axis, in.
y_1, y_2	Vertical distance from designated edge of member to center of gravity, in.
z	Coefficient for buckling of triangular-shaped bracket plate
Δ	Deflection, in.
α	Fraction of member force transferred across a particular net section
α	Ratio of moment at bolt line to moment at stem line for determining prying action in hanger connections
α	Ideal distance from face of column flange or web to centroid of gusset-to-beam connection for bracing connections and uniform force method, in.

$\bar{\alpha}$	Actual distance from face of column flange or web to centroid of gusset-to-beam connection for bracing connections and uniform force method, in.
β	Ideal distance from face of beam flange to centroid of gusset-to-column connection for bracing connections and uniform force method, in.
$\bar{\beta}$	Actual distance from face of beam flange to centroid of gusset-to-column connection for bracing connections and uniform force method, in.
δ	Deflection, in.
δ	Ratio of net area at bolt line to gross area at face of stem or angle leg used to determine prying action for hanger connections
ε	Coefficient of linear expansion, with units as indicated
τ_a	Stiffness reduction factor, for use with the alignment charts (AISC Specification Figures C-C2.3 and C-C2.4) in the determination of effective length factors, K, for columns
ν	Poisson's ratio = 0.3 for steel
ϕR_n	Design strength from AISC Specification; must equal or exceed required strength using LRFD load combinations, R_u
ϕr_n	Design strength per bolt or per inch of weld from AISC Specification; must equal or exceed required strength per bolt or per inch of weld using LRFD load combinations, r_u
R_n/Ω	Allowable strength from AISC Specification; must equal or exceed required strength using ASD load combinations, R_a
r_n/Ω	Allowable strength per bolt or per inch of weld from AISC Specification; must equal or exceed required strength per bolt or per inch of weld using ASD load combinations, r_a

INDEX

The following list of terms provides reference to items found in the AISC *Steel Construction Manual*, as well as selected supporting references. The locations of supporting references have been abbreviated as follows:

"DG#" is used for items found in AISC's Design Guide series.
"SDM" is used for items found in the AISC *Seismic Design Manual.*
"DSC" is used for items found in AISC's *Detailing for Steel Construction.*
"AISC Design Examples" indicates that information can be found on the CD companion to this AISC *Steel Construction Manual.*

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